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Paper No. 1379

DESIGNING AN EARTH DAM HAVING A GRAVEL FOUNDATION, WITH THE RESULTS OBTAINED IN TESTS ON A MODEL*

By James B. Hays, Jun. Am. Soc. C. E.

With Discussion by Messrs. W. G. Bligh, J. C. Oakes, C. E. Grunsky, H. T. Pease, Malcolm Elliott, Edward Wegmann, E. C. La Rue, George M. Bacon, H. A. Petterson, D. C. Henny, Joseph Jacobs, and James B. Hays.

Synopsis.

The following paper, though giving the theory regarding the design of an earthen dam having a gravel foundation, is intended mainly to present the results of tests on a model constructed to scale. The dam was designed to impound water for storage purposes under rather unusual conditions. A study of the subject produced little of real value in determining the action of water under these conditions.

However, this paper does not give the detailed design of unimportant features but only the general design, especially the shape and type of cross-section and the quality of the materials placed therein.

It is hoped that the discussion will bring out many interesting facts and experiences which will contribute to more rational methods of design of earth dams, or of any dams, on porous foundations.

Introduction.

At the dam site in question, a dam had been started, but was wholly unfit to sustain even a small proportion of the head of water

* Presented at the meeting of April 19th, 1916.

required. The reservoir was for the storage of water for irrigation purposes, and the dam was to be 110 ft. high at the maximum section. With an approximate depth of 30 ft. of material in place, and nearly the whole width of the base, a head of slightly less than 25 ft. caused water to issue in considerable quantities from the down-stream toe of the dam. This caused the people living in a small town a few miles down the river to become fearful of the results.

Several reports, made by different engineers, unanimously condemned the structure as it was being built. The bonding house backing the irrigation company failed at this time, the project was sold to satisfy judgments, and the contractors for the dam took over the proposition.

At their request, F. C. Horn, M. Am. Soc. C. E., was asked to submit a design for a dam to be built at the same site, if it could be done, in order to utilize as much of the existing structure as possible. On account of the unusual conditions—which will be stated later—it was decided to make a thorough research, to be followed by experiments and tests, in order to make certain of the design. The writer did much of the research work, constructed the model, and made the tests for, and with, Mr. Horn, which the latter has kindly made available.

General Conditions.

The stream on which the dam was being built runs southeastward, and, on the southwest side of the valley there are high cliffs of cherty limestone near which the stream has its course. On the opposite side of the valley there are high mountains at some distance from the river. Mountain streams, rushing along at high velocity, had carried gravel, sand, and some boulders out into the valley bottom, where the material was dropped, forming great cones, or fans, of very porous material. The dam site had been originally selected having one of these gravel cones as the foundation and northeast bank, this being done with only a superficial knowledge of the subsurface conditions.

When trouble was reported, work was ordered stopped by the State Engineer; then a few borings were made and test pits dug, the results of which were available for the re-design of the structure. The borings, which were made close to the center line, or axis, of the dam, show that it would be practically impossible, and useless, to reach bed-rock throughout the entire length of the dam and thereby cut off the underground flow of water.

In the subsoil investigations, a material was encountered which was called hardpan. Chemical investigation shows that there is practically no clay in this material, and, in fact, no clay in the vicinity. The material called hardpan was very fine, gritty, and angular. It was tightly packed, and formed a very dense mass.

A large quantity of this fine material was found on the surface, about a mile from the dam, and when shaken down in a glass container, packed into a very tight, dense mixture. This material is often called hardpan, but is not real hardpan, as it contains no clay.

A cut-off wall of concrete was built across the river channel, beneath which steel sheet-piling was driven into this so-called hardpan. Some of the piling was driven 18 ft. deep, and other portions sank as deep as 32 ft. with one or two blows from a 1 700-lb. hammer.

The body of the dam was being built of the same material as that composing the gravel cone at its northeast end, and, as was shown in one of the reports, nearly all the water, except flood waters, of the creek that had con-

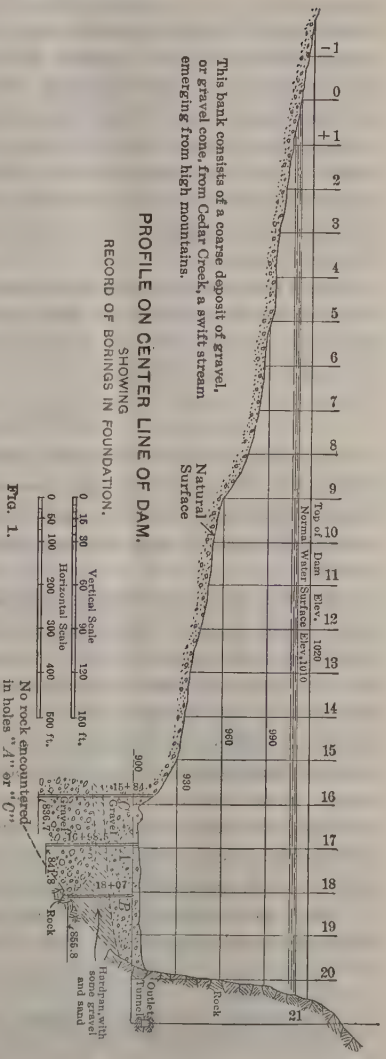


FIG. 1.

structed this cone was sinking into the gravel and reappearing at a lower point. This, it would seem, should have been an object lesson to the original locators of the site, and would indicate the conditions as to its character. However, little attention seems to have been given to the matter, and a dam was being built to hold up a head of 105 ft., to have a top width of 20 ft., with slopes of $2\frac{1}{2}:1$ up stream, and 2:1 down stream. The down-stream toe had a gravel or loose rock drain, and the up-stream face had a concrete facing, which, for some unknown reason, did not extend to the toe of the dam.

Fig. 1 shows the results of the borings on the center line of the dam. Thus, it will be seen that the impervious stratum is a very uncertain quantity, and that bed-rock is very deep and practically impossible to reach with a cut-off wall.

General.

In looking for precedent in the design of a dam under these conditions, Mr. Horn and the writer were confronted with a lack of good material based on a systematic study of structures of this type, although several good points were obtained from some of the papers on the subject.

As these tests, experiments, and designs were made during the period from August, 1914, to August, 1915, the valuable information contained in the paper by J. B. T. Colman, Assoc. M. Am. Soc. C. E., on "The Action of Water Under Dams",* was not available. Although Mr. Colman was unable to effect a complete loss of head with his model, the writer was successful in this, and has explained the difference in construction of the two models in his discussion† of that paper, which accounts for the different results obtained. However, although the writer's experiments cover the design of the dam, as well as the study of the flow of the underground water, he believes there is much material of a different nature contained in this paper.‡

The paper§ entitled "The Bohio Dam", by the late George S. Morison, Past-President, Am. Soc. C. E., contains some interesting

* *Transactions*, Am. Soc. C. E., Vol. LXXX, p. 421.

† *Ibid.*, p. 458.

‡ In Professional Memoirs, U. S. Engineer Corps, Vol. VII, p. 44, par. 33, there is a record of an experiment similar to a part of Mr. Colman's.

§ *Transactions*, Am. Soc. C. E., Vol. XLVIII, p. 235.

points, and the discussion on that paper brings out some good details regarding the design of the North Dike of the Wachusett Reservoir. In this structure a deep trench was excavated parallel to the axis of the dam, and in the bottom of this trench wooden sheet-piling (some pieces as long as 60 ft.) was driven. Percolation and seepage experiments were conducted with a tank similar to the one used and described by Allen Hazen, M. Am. Soc. C. E., in his percolation experiments. In the case in hand, a tank of the same style was adopted for the preliminary experiments, and furnished much valuable information to be used in the final tests. A large wooden tank was also constructed for the purpose of studying the probable percolation and seepage through the structure. In this case (the North Dike of the Wachusett Dam), the foundation, or subsoil, was not considered in the tests; thus the conditions differed from those which had to be considered in the design of the dam described in this paper.

In the North Dike there was a long, rather flat, down-stream slope, and a steeper up-stream slope. A flat hydraulic gradient, caused by the water in the reservoir seeping under the dam, called for a large quantity of material, in order to withstand the upward pressure under the down-stream portion of the dam. The dam was of such a length that the water, rising from the down-stream toe, was under no upward pressure and flowed away without disturbing the soil.

The Gatun Dam, built in connection with the Panama Canal, was designed and constructed under similar conditions, having a silty river deposit for a foundation. A flat down-stream slope causes the percolating water to travel a long distance before a free opening is encountered, thus causing the upward pressure to be consumed by friction.

In discussing the paper entitled "Dams on Sand Foundations: Some Principles Involved in Their Design, and the Law Governing the Depth of Penetration Required for Sheet-Piling",* G. E. P. Smith, M. Am. Soc. C. E., works out a very interesting theory,† showing that the creep, or total distance which the water would travel, would be increased by an amount equal to twice the length of the sheet-piling introduced. It is not stated in so many words, but the loss of velocity in traveling down one side of the sheet-piling and up the other side

* *Transactions*, Am. Soc. C. E., Vol. LXIII, p. 175.
† *Ibid*, p. 197.

amounts to the same as adding twice the length of the sheet-piling to the length of the base.

W. G. Bligh, M. Am. Soc. C. E., gives a very interesting treatment of the design of weirs on sand foundations.* His theory is that the water will follow the line of creep, along the base of the dam, down one side of the cut-off wall, up the other side, and continue along under the base to a free outlet, that is, that pressure is lost in direct proportion to the enforced line of creep. This theory gives structures of ample dimensions, as has been shown by practice, but was found to be incorrect, after due experimenting, as will be noted later.

E. A. Moritz, Assoc. M. Am. Soc. C. E., has given† the customary method followed by the United States Reclamation Service Engineers. This, it will be noted, follows the line of creep theory as described by Mr. Bligh.

D. C. Henny, M. Am. Soc. C. E., made tests‡ on different combinations of the available local materials in order to determine the tightest mass. Tests for seepage were made with many different mixtures.

In the writer's method, only one combination of materials was made, and that was done by a study of the sieve analysis curves.§ Such curves are plotted for the various materials at hand, and compared with the ideal curve for the densest mass. Experience has proved that this method gives a very tight concrete. The experiments described in Mr. Henny's paper did not cover the seepage under the dam, but adequate drainage was provided in order to forestall the danger of the down-stream bank sloughing off.

Experiments and Tests.

The foregoing sources of information are mentioned in order to show what materials were available, to give credit where similar plans were followed, and, for the sake of comparison, to state where results differed, which was largely the case.

* "Practical Design of Irrigation Works", Chapter VI.

† "Working Data for Irrigation Engineers", pp. 39–40.

‡ "Two Earth Dams of the United States Reclamation Service", *Transactions*, Am. Soc. C. E., Vol. LXXIV, p. 38.

§ This method is explained in Taylor and Thompson's "Concrete, Plain and Reinforced", pp. 194, *et seq.*

The first experiments were for the purpose of determining the loss of head due to the water percolating through the various materials.

In the first test the attempt was made to determine the hydraulic gradient, or loss of head, through the coarse material constituting the foundation of the dam, and of which a part of the structure was already built. The tank had a diameter of 30 in., and a total height of 5 ft. The inside of the tank was given two coats of paint, and was then sanded in order to prevent possible seepage along its walls. Two glass tubes extending upward on the outside of the tank were connected with the inside and arranged to indicate the loss of head between two points which were vertically distant 3 ft. Fig. 2 shows the tank. A valve at the bottom held back the water until the pore spaces were completely filled; then the valve was opened, and after a few minutes the relative elevation of the water in the two tubes became constant. The difference in elevation determined the loss of head, and from this the hydraulic gradient was computed.

The initial run gave a very small loss of head; a few days later, the loss was greater and remained constant. This is explained by the fact that the material was placed in the tank in a dry state and the first application of water readjusted the finer materials into what in all probability represents the natural condition. With 6 in. of water on the soil surface, and 8 in. of soil above the upper tube, the loss of head was 1 ft. in 9.

This gravel was screened into its various sizes, and showed a very small percentage of fine material. In the upper diagram on Plate I, Curve *A* shows this material as plotted. Fig. 3 is reproduced from a photograph of a sample of this material, and Fig. 4 shows the material as it stood in the steam-shovel pits from which the dam was being constructed and from which point the sample was taken.

Having determined the hydraulic gradient of the underground material, a trial design was made to find what dimensions would be necessary in a dam constructed wholly of this gravel. The lower diagram on Plate I shows the dimensions, assuming that the hydraulic gradient began at the water surface on the up-stream face of the dam. The effect of the core-wall on the "line of creep" theory would be small. The section shown was deemed to be very inefficient and excessively expensive, on account of the large quantity of material

to be placed. Further investigations were then made to determine how the section could be reduced.

Following out the "line of creep" theory as the one which seemed to be more nearly correct, on account of its practical success, it was decided that, if a tight material could be obtained for the up-stream section, the hydraulic gradient could be forced to begin farther up stream, and thus there would be secured the advantage of the up-stream section in reducing the upward pressure before the water reached the portion of the dam below the center line or core-wall.

As mentioned previously, a large body of very fine material was available in the vicinity. A sample of this material when analyzed gave Curve *B* in the upper diagram on Plate I. Curve *A* showed a lack of material, which was just balanced by the excess of fine material in Curve *B*. By observation, it was determined that, for all practical purposes, the combination of these two materials in the ratio of 1:1, would give a very dense mixture which would approach closely the ideal curve. Curve *C* represents the combined material. Fig. 5 shows a sample of the fine material, and Fig. 6 shows the combined material.

TABLE 1.—Mechanical and Chemical Analyses of the Fine Material Obtained One Mile from the Site of the Proposed Dam.

Mechanical Analysis. This analysis covers only the material remaining on, and passing, the 30-mesh sieve.			Chemical Analysis. Air-dried sample.	
Mesh.	Character.	Percentage, by weight.	Constituents.	Percentage, by weight.
+ 30	Sand	7.86	Woody fiber	0.52
+ 60	"	10.06	Insoluble	52.16
+ 100	"	19.20	Silica (SiO_2)	44.48
+ 200	"	21.27	Ferric Oxide (Fe_2O_3)	2.46
— 200	Fine sand and silt	40.00	Alumina (Al_2O_3)	5.92
— 200	Clay	1.61	Lime (CaO)	18.27
			Magnesia (MgO)	5.70
			Water at 110° cent	0.60
			Loss on Ignition (CO_2), Organic, etc.	21.58
			Undetermined	0.99
		100.00		100.00

This combined material was then tested in the tank to determine the hydraulic gradient. The head of water in the tank above the soil immediately came to a maximum, and showed only a very slight fall

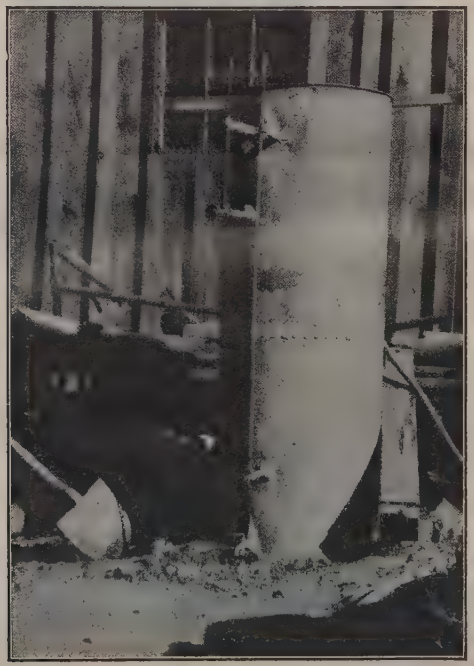


FIG. 2.—VERTICAL TANK FOR PRELIMINARY TESTING OF GRAVEL AND SOIL

FIG. 3.—NEAR VIEW OF GRAVEL IN NATURAL POS[illegible]

FIG. 4.—STEAM-SHOVEL PITS FROM WHICH COARSE GRAVEL WAS TAKEN.

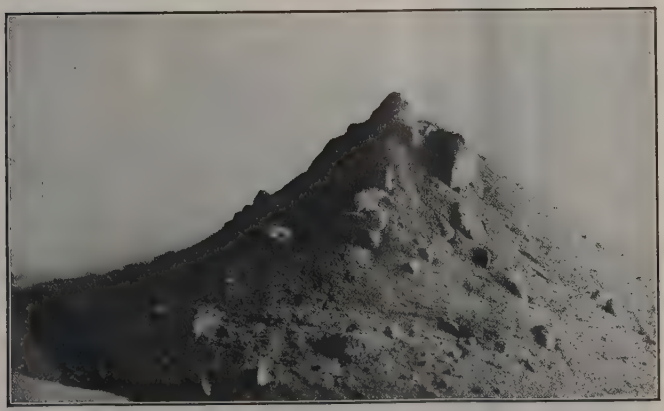


FIG. 5.—SAMPLE OF FINE MATERIAL USED TO COMBINE WITH THE COARSE GRAVEL TO OBTAIN A DENSE MIXTURE.

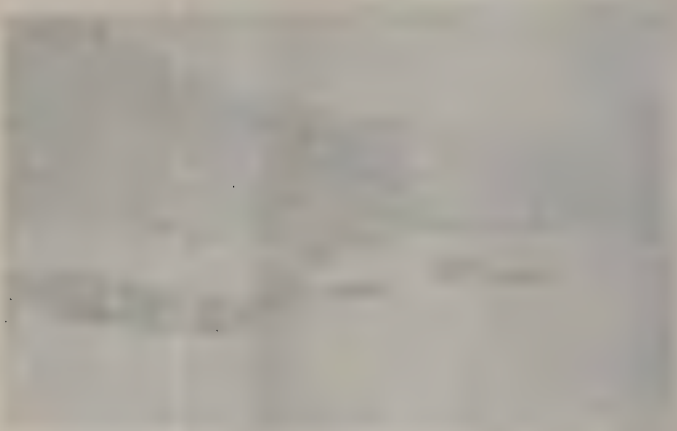


FIG. 6.—COMBINED COARSE AND FINE MATERIALS, MIXED IN EQUAL PARTS OF EACH.

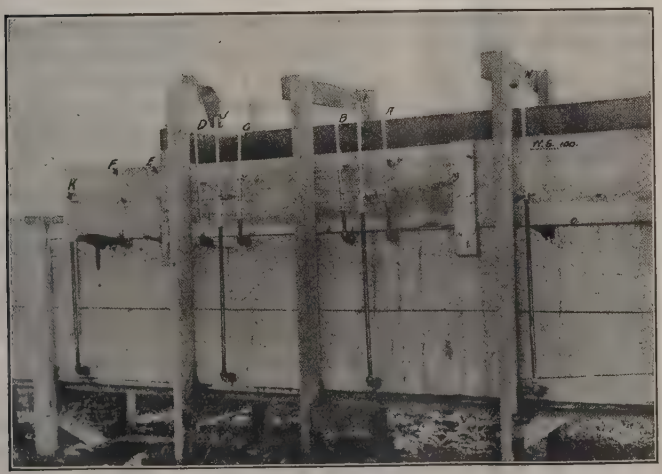


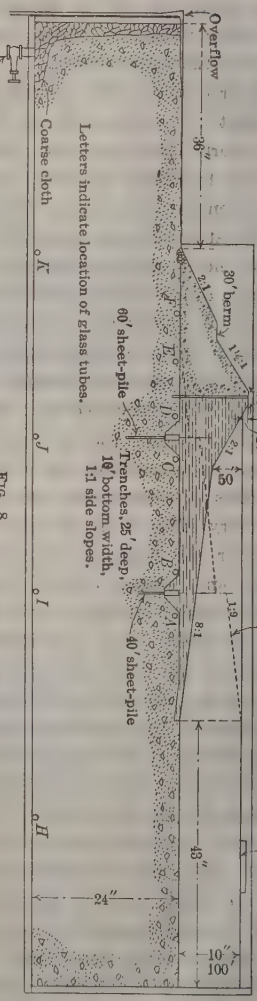
FIG. 7.—TANK IN WHICH FINAL EXPERIMENTS WERE CONDUCTED. FULL HEAD OF WATER IN TANK WHEN PHOTOGRAPH WAS TAKEN.

during the first day. In the morning of the second day, fully 80% of the water was still on the surface, and at the end of 47 hours both tubes were filled with water, though the valve at the base had been closed for the entire period.

On proceeding with the tests it was found impossible to determine the exact hydraulic gradient, as no water appeared in either of the tubes shortly after the valve was opened. From this it was assumed that the hydraulic gradient was not greater than 1:1, although it was evidently much steeper. Hence the design was made on this basis.

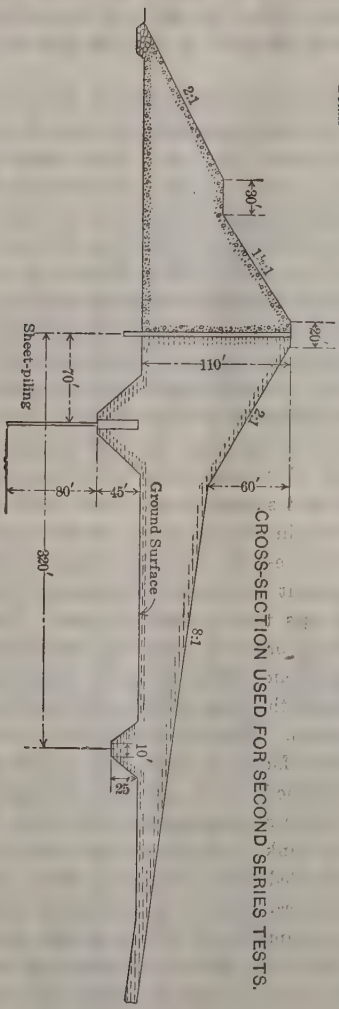
This structure was designed by following out the "line of creep" theory, and it was calculated that, for a head of 100 ft. of water in the reservoir, a line of creep of 900 ft. would be necessary at the maximum section. By using the combined materials in making the up-stream section, in order to secure water-tightness, and avoid the transmission of the water pressure through the up-stream section in any great amount, it was assumed that the hydraulic gradient could be made to begin at or near the up-stream toe of the dam. Then, by the introduction of sheet-piling, the length of the base could be determined. Two rows of sheet-piling were at first suggested, and these could be made very effective by driving them in the bottom of trenches excavated for the purpose. Concrete core-walls over the piling would provide an adequate cut-off to prevent the water from passing over it. The design shown in Fig. 8 was then prepared for test. The long up-stream slope was given in order to allow the downward pressure of the water over the up-stream section to have a balancing effect on the upward pressure beneath the dam, as blow-outs would be improbable in this portion of the dam. This also gave a drier down-stream section, which meant a larger factor of safety from sloughing off and the like. A theoretical grade line is shown applied to the design.

Then, for the final tests, a model was constructed to the scale of 1 in. to 10 ft., and a large wooden tank of the dimensions shown in Fig. 8. All joints were caulked and filled with a composition filler, and then the whole interior was given two coats of a water-proof roof paint. On the second coat sand was applied, in order to prevent possible leakage along the walls. The glass tubes were placed at the points indicated by letters. They communicated by pipes with the interior of the mass at two points, and were set in from the sides of the box in order to avoid percolation along the walls. At the down-



TRIAL DESIGN IN TESTING TANK
SCALE OF MODEL, 1″ = 10′ ACTUAL.

FIG. 8.



CROSS-SECTION USED FOR SECOND SERIES TESTS.

FIG. 9.

stream end of the tank there was an overflow and also a drain. A width of 4 in. at the end of the box was filled with very coarse gravel, and this was separated from the remaining gravel by a coarse cloth. This allowed the water to travel in straight lines to the end of the box, and did not tend to force the water up to the ground surface unnaturally.

Several of the glass tubes were intended to indicate the upward pressure of the water on the base of the dam, and the others were placed along the bottom of the box to show the influence of the sheet-piling on the flow of water at the deeper points.

From the information at hand it was assumed that the bed-rock would not be more than 240 ft. below the base of the dam under the maximum section. The up-stream toe of the dam was then placed about 43 in. from the head of the box. In this way there would be plenty of room for the water to get into the soil; in other words, it was deemed necessary to avoid a narrow section or constriction through which the water would have to be forced, thereby causing a greater loss of head. A space of 3 ft. was left on the down-stream side of the dam to permit the water to come up to the surface; this was thought to be sufficient when the vertical loose-rock drain was allowed at the end.

All the subsoil material was placed dry, which made it necessary to disregard the first few readings. More than thirty readings were taken, and consisted of several runs, at four or five different heads above the dam. Records were kept of the quantity of water seeping through the gravel bed; these are given in the last column of Table 2.

Fig. 10 shows typical results obtained in the first series of tests. It will be noted that the loss of head does not follow the rule regarding the line of creep. This is shown particularly at the upper row of sheet-piling, where the loss of head is very small and not in proportion to the length of the sheet-piling. At the lower row of sheet-piling, however, near the center line of the dam, the loss of head is far greater in proportion than that at the upper row.

According to the theoretical hydraulic grade line shown on Fig. 8, the loss of head due to the upper row of sheet-piling would be, following the "line of creep" theory, 7.2 ft., though it was actually from 3.5 to 5 ft.. only. The theoretical loss at the second or longer row of sheet-piling was calculated to be 9.4 ft., and the actual loss was more than twice as much, or from 19 to 21 ft.

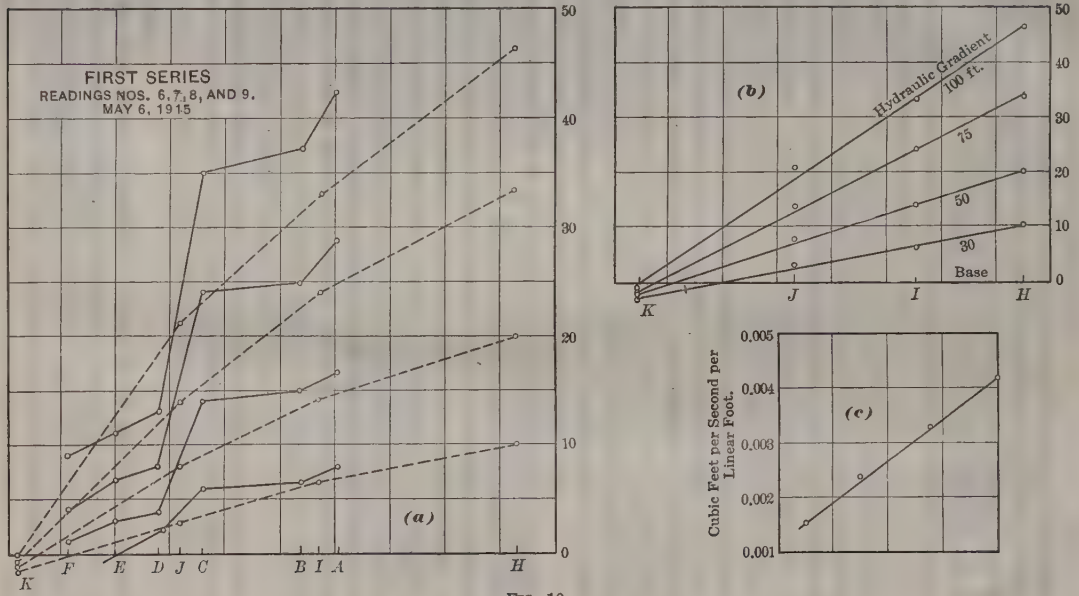

FIRST SERIES
READINGS NOS. 6, 7, 8, AND 9.
MAY 6, 1915
(a)
K
F
E
D
J
C
B
I
A
H
0
10
20
30
40
50
(b)
Hydraulic Gradient
100 ft.
75
50
30
Base
K
J
I
H
0
10
20
30
40
50
(c)
Cubic Feet per Second per Linear Foot.
0.001
0.002
0.003
0.004
0.005

FIG. 10.

TABLE 2.

No. of reading.	Date, 1915.	Time.	Head.	*A*	*B*	*C*	*D*	*E*	*F*	*H*	*I*	*J*	*K*	*S**
1	5–4	10 A. M.	100	.....	44	41	16	14	11	55	41	26	— 2	.....
2	5–5	9.30	50	.....	18	17	5	4	2	23	17	10	— 2	0.00187
3	"	"	75	.....	29	28	10	8	5	37	27	17	0	0.00235
4	"	"	90	.....	36	34	13	10	8½	44	33	20	½	0.00395
5	"	"	100	.....	41	39½	15	12	10	52	38	24	1½	0.00445
6	5–6	10.00	80	8	6½	6	2	0	0	10	6	3	— 3	0.00151
7	"	"	50	17	15	14	4	3	1	20	14	8	— 2	0.00237
8	"	"	75	29	25	24	8	7	4	33	24	14	— 1	0.00323
9	"	"	100	42	37½	35	13	11	9	46	33	21	0	0.00414
10	5–7	10.00	25	6	4½	4½	0	0	— 1	8	15	2½	— 3	0.00142
11	"	"	50	16	14	13	3½	2½	0	18	13	7	— 2	0.00323
12	"	"	75	28	24	22½	8	6	4	31	23	14	— 1	0.00310
13	"	"	90	35	31	29	10	9	6	39	28	17	0	0.00374
14	"	"	100	40	35	34	12½	10	7½	44	33	20	1	0.00395
15	"	2.20	25	6½	5	5	0	0	— 1	8½	5	3	— 3	0.00142
16	"	"	52½	17	15	14	4	3	2	19½	14	8½	— 2	0.00237
17	"	"	77½	26	23	22	8	6	3	30	21	13	— 1	0.00309
18	"	"	90	30	27	26	10	7½	5	34½	25	15½	0	0.00356
19	"	"	100	34½	31	30	11	8	6	40	29	18	0	0.00374
20	5–21	10.00	30	11	11	8	1	0	0	12	8	3½	— 5	0.00151
21	"	"	50	20	19	17	4½	2	0	22	14	9	— 3	0.00287
22	"	"	75	35	34	28	20	8	5	37	28	16	— 2	0.00323
23	"	"	90	44	41	35	14	10	7	46	34	21	— 1	0.00395
24	"	"	100	50	47	40	15	11½	9	54	39	23½	0	0.00414
25†	"	"	80	41	37	36	11	9	6	43	30	18½	— 1	.....
26†	"	"	50	22	19	17	5	3	1	23	15	9	— 3	.....
27†	"	"	30	11	10	8	2	0	0	12	8	4	— 4	.....
28	5–25	"	32	11	10	8½	0	0	0	11	8	4	— 4	0.00114
29	"	"	50	23	20	18	4½	0	0	23	16	10	— 3	0.00216
30	"	"	75	28	‡	28	9½	6	2	37	27	15½	— 1½	0.00299
31	"	"	90	‡	‡	41	36	13	8	5½	46	34	20	— 1	0.00340
32	"	"	100	‡	‡	47	40	14	10	?	52	39	23	0	0.00374

* This column gives the seepage or percolation, in cubic feet per second per linear foot of model dam.

† Experiments were taken with water receding in order to compare with others; also, no percolation readings were taken.

‡ Figures excessive, due to water breaking through up-stream section of model dam.

NOTE.—The figures in this table may be converted to inches by pointing off one place and reading direct, thus 17 equals 1.7 in.

The writer wishes to state at this point that where the figures were given in feet, for convenience in stating the case relative to the actual structure, they represent actually tenths of inches on the model, that is, 19 ft. as stated above was 1.9 in. measured on the model. The figures given in the preceding paragraph were for a full head (10 in. equals 100 ft.).

The hydraulic grade line, represented by the upper row of tubes, indicated that there was an appreciable loss of head at the down-stream toe of the dam, and the lower row of tubes showed a complete loss of head at Tube *K*.

Fig. 10 (*b*) shows the hydraulic grade line of the lower row of tubes when the distance is computed from the up-stream toe of the

dam to each tube directly, and it will be noted how close the hydraulic gradient is to a straight line. Tube *J* was evidently affected by the sheet-piling to some extent.

Fig. 10 (*c*) shows the seepage loss from the body of water above the dam. This subject will be discussed later.

The failure of the "line of creep" theory to work out on the model as it was expected, caused the development of another theory as to the effect and usefulness of sheet-piling as a cut-off, preventing or reducing the underground flow of water in dams designed or built under similar conditions.

The theory finally evolved was that the sheet-piling cut-off was greatly similar in its effect to a partly closed valve, wherein the water is retarded and shows a higher pressure head just above the cut-off and a lower one just below. This theory is well known in regard to the hydraulic gradient of a pipe line, and explains the inefficiency of the upper row of sheet-piling as a cut-off. Hence, in the final design of the dam, this upper or shorter row of sheet-piling was omitted, and the second row was lengthened.

The model was then reconstructed according to this idea, making the piling 12½ in. (125 ft.) instead of 8½ in. (85 ft.). Tests were then made in exactly the same manner as before, and the results obtained showed a very noticeable effect due to the increased length of sheet-piling, which bore out the theory as to effect of sheet-piling and its similarity to the partly closed valve.

Fig. 11 shows typical results taken from the last series of tests, as in Table 3. Fig. 11 (*a*) shows a graphical record of the readings taken on the tubes. It will be noticed that the hydraulic grade line cuts the base of the dam well up stream from the toe. Fig. 11 (*b*) shows the hydraulic grade line of the lower set of tubes, where the distances from the up-stream toe of the dam to each tube were plotted horizontally, to show more correctly the hydraulic grade line of the water flowing through the gravel beneath the dam. Fig. 11 (*c*) shows the seepage, in cubic feet per second, per linear foot of the model.

Fig. 7 shows the tank for testing the model, under a full head. The water was colored red in order to show clearly in the photograph.

The down-stream section of the dam was constructed of the same material as the foundation, in order to provide adequate drainage and

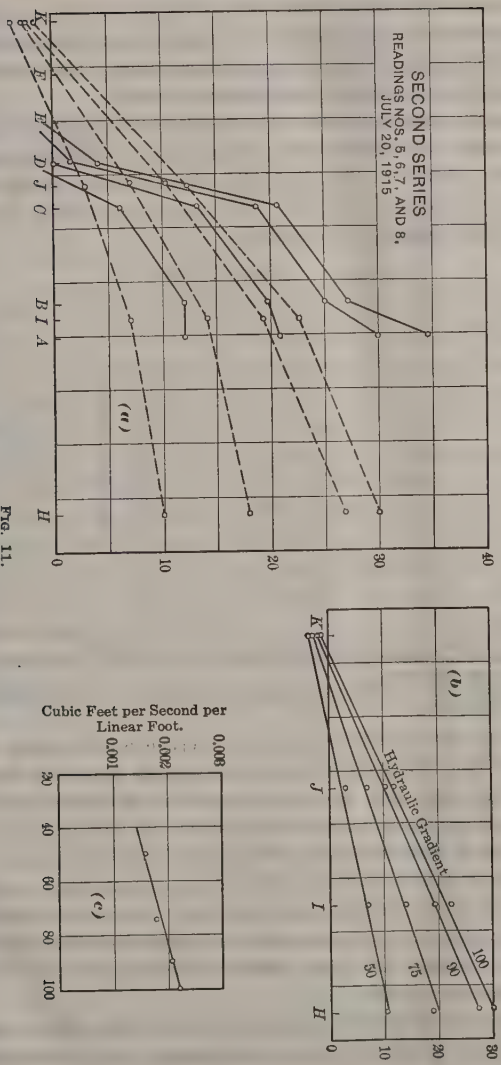

SECOND SERIES
READINGS NOS. 5, 6, 7, AND 8.
JULY 20, 1915
(a)
K
G
F
E
D
J
C
B
I
A
H
0
10
20
30
40
(b)
K
J
I
H
0
10
20
30
Hydraulic Gradient
100
90
75
50
Cubic Feet per Second per Linear Foot.
0.001
0.002
0.003
(c)
20
40
60
80
100

Fig. 11.

for economy. It will be noted, from the plotting of the readings of the upper row of tubes, that there was no water in this section of the dam, except possibly that due to capillary action. This contributes a great deal to the stability of the dam as a whole.

It will be noted from Figs. 10 and 11, that the water traveling at great depths is not materially affected by the sheet-piling, at any particular point; but, throughout the whole distance under the dam, the general effect is obtained. This may be attributed to the extremely slow velocity of the water.

TABLE 3.—SECOND SERIES (*a*).

No.	Date, 1915.	Time.	Head.	*A*	*B*	*C*	*D*	*E*	*F*	*H*	*I*	*J*	*K*	*S**
1	7–19	10 A. M.	50	16	13	8	0	0	0	14	9	4	—4	0.00142
2	"	"	75	27	22	16	0	0	0	23	17	9½	—3	0.00202
3	"	"	90	33	25	21	4	0	0	30	23	12	—2	0.00237
4	"	"	100	40	30	25	6	2	0	35	26	15	—1½	0.00304
5	7–20	"	50	12	12	6	0	0	0	10	7	3	—4	0.00121
6	"	"	75	21	19	13	0	0	0	18	14	7	—3	0.00174
7	"	"	90	30	25	18½	2	0	0	27	19	10½	—2½	0.00208
8	"	"	100	34½	27	21	4	0	0	30	22½	12	—2	0.00217
9	7–21	"	55	15	8	6	0	0	0	11	7	3½	—4	0.00127
10	"	"	75	23	15	12½	1	0	0	19	13	7	—3	0.00170
11	"	"	91	31	22	17	1	0	0	26	19	10	—3	0.00203
12	"	"	100	44	26	20	3	0	0	30	21	12	—3	0.00229
13	9–16	2 P. M.	30	10½	0	0	0	0	0	—3	—4	—4	—6½	0.000418
14	"	"	50	23	7	0	0	0	0	2	1	—1	—5	0.000857
15	"	"	75	40	22	7	0	0	0	8	5	3	—4½	0.00142
16	"	"	90	†	30	13	0	0	0	14	9	5	—3	0.00182
17	"	"	100	†	36	16	6	0	0	20	13	8	—3	0.00222

* Percolation, in cubic feet per second per linear foot of model.

† Readings for Tube *A* were excessive, due to breaking through the thin section of model dam over that tube.

Minus readings show that the pressure, as indicated by the tubes, did not reach the base of the dam.

Figs. 10 (*c*) and 11 (*c*) show the seepage water coming through under the dam. The results show that seepage water will follow the straight-line rule, after the soil is once filled with water and a sufficient head has been reached to overcome the friction. The rate of increase of flow, for each additional unit of head applied, varies for sands and gravels under different conditions, such as porosity of sand or gravel, character of voids, etc. As it would take a very extensive set of experiments to make any determinations in regard to the laws of seepage, the writer merely wishes to present the results so that others may use them.

In any dam, the correct place for a cut-off wall, under conditions such as these, namely, a pervious foundation, is at, or near, the up-

stream toe. However, in an earth dam, it must of necessity be placed nearer to the center, or axis, of the dam. For construction reasons, the sheet-piling in this dam was placed a short distance up stream from the center line, but, at the same time, it was sufficiently far back to prevent any water from seeping through the dam to the down-stream side of the piling in any appreciable quantities.

Conclusions.

Earth dams fail from four main causes, and the writer desires to show how structures of this type may be made safe.

One cause is from overtopping, due to high waves or a flood. This is provided against by designing an adequate spillway and providing sufficient free-board. This point has not been discussed in this paper, as it is governed by local and not general conditions. The concrete core-wall was designed to extend to the top of the embankment, which would prevent waves from cutting through.

A second cause of failure is by water seeping along a conduit placed in the built-up portion of the dam. In the present case, the outlet consists of a tunnel through the rock cliffs at one end of the dam, thus eliminating this cause of failure.

A third cause, and an unseen one, is burrowing animals. Where the down-stream section is loose rock this cannot happen, and it is much the same in loose gravel.

The fourth cause is the occurrence of springs, or boils, on the down-stream face of the dam, or near it, which in turn cause blow-outs and sloughing off of the bank. A solution of this problem was the main object of these experiments. Springs, or boils, are caused by not consuming all the head of water above the dam. From the tests described herein, it is seen that either an extreme length of base of the dam must be provided, or there must be sheet-pile cut-off walls of the proper length. Where the sand or gravel foundation is of very great depth, a short cut-off would not be as efficient as in a shallower foundation, but where models are constructed, results can be obtained which enable the engineer to make correct designs better than by any other method of study.

Finally, the writer wishes to state that he claims no new discovery, but has determined a type that has solved the problem under the conditions with which he was confronted. This type has been

built before, but has not been advanced together with reliable information from tests, etc. The study of the effect of the sheet-piling will undoubtedly be of value to the Profession. The application of the sieve analysis curves in combining the materials, for the purpose of determining the densest mass obtainable, has not been advanced before—as far as the writer knows—but its value as a time saver can easily be seen. Levees, canal banks, dikes, etc., may be studied in the same manner. Each design is a problem in itself, and the experiments described herein should not be misinterpreted, or applied too broadly.

DISCUSSION

Mr. Bligh.

W. G. Bligh,* M. Am. Soc. C. E. (by letter).—This paper is of exceptional interest, as it deals with the first actual experiment made in many years with the object of ascertaining the value of the reduction in pressure or head caused by vertical obstructions such as curtain-walls or diaphragms of sheet-piling projecting below the base of a dam.

Any experiments with models, to be of authoritative value, must be on a reasonably large scale, and, further, the conditions should not deviate from those actually encountered in practice. In the present case, the scale of the model is considered too small, and might well have been 10 ft. to 1 in. However, the experiments in question were undertaken by private parties for a specific purpose, and the results obtained are sufficiently reliable for the aim in view. It is sincerely to be hoped that this good work may be continued, and this much discussed point finally disposed of in a convincing and indisputable manner. With regard to the second suggested condition, the experiment is all that could be wished, the relation between the head of water and the length of base provided in the model being the same as would be the case in the actual dam when constructed. This condition has not been observed in the paper by J. B. T. Colman, Assoc. M. Am. Soc. C. E., on "The Action of Water Under Dams",† where the disproportion between the head of water and the length of travel provided for the percolating undercurrent is unreal and, in a measure, must vitiate the authoritative value of the deductions obtained.

In the model dam, the base length was made dependent on actual experimental data obtained by filtration. This determined the net value of base length or travel of the percolating current requisite to neutralize entirely the head of water. It is quite evident that the length thus obtained must in many cases be a maximum value, whereas the minimum safe base length is the real criterion. Supposing the base to be shortened, the neutralization of the whole head would then still take place, but the velocity of the percolating undercurrent would be increased, as well as its ability to wash out and carry away particles of sand or gravel from under the dam and thus threaten piping action, which, in time, would cause undermining. This minimum length of travel suitable to any description of substratum could be obtained by actual experiment to which a factor of safety would necessarily be applied of, say, not less than 1.3. It is possible that, in certain cases, the base length thus deduced would nearly equal the maximum, but in nearly all instances it will be very much less. For

* Toronto, Ont., Canada.

† *Transactions*, Am. Soc. C. E., Vol. LXXX, p. 421.

Mr. Bligh.

example, in the extreme case of a loose boulder river bed, the length requisite to neutralize completely a given head of water by filtration would be altogether impracticable; on the other hand, owing to the weight of the individual particles composing the foundation, a comparatively high velocity in the undercurrent would be admissible with a correspondingly reduced length of base. As far as the writer is aware, no comprehensive experiments have ever been undertaken to solve this matter in a really satisfactory manner.

The so-termed "percolation factors", first introduced some years ago by the writer, were deductions from actual failures of certain large river works, and although they are undoubtedly reliable for beds of pure sand, and have been accepted as such, still further definite experimental data would be most desirable.

This experiment demonstrates that the loss of head due to the vertical diaphragms is proportionally less in the rear than in the fore sheet-piling, and that, in neither case, is this neutralization in agreement with the so-termed "creep" theory. The author attributes the great difference in the effectiveness of the two lines of sheet-piling to the much greater depth of fore piling as well as to its location half way down the base of the dam, the assumption being that the obstruction of the deep curtain in the center of the base acts as a throttle valve to the percolating subcurrent at a position where the latter is well under way, thus causing a sudden loss of head. This view the writer deems to be correct, but applicable only to the quite abnormal conditions present in the experimental model. The depths given to the sheet-piling are far in excess of actual practice. That of the central line reaches to 125 ft. below the base of the dam, that is, to half the depth of the porous stratum; half the waterway is thus cut off, which must cause congestion. It is quite possible that this throttle-valve action may exercise a retroactive effect on the rear diaphragm, reducing its efficiency. However this may be, it is evident that the abnormal conditions existing in the model render the results obtained from the experiment of small practical value, and that the actual neutralizing effect of vertical curtain-walls remains still to be decided by further experiments made under normal conditions.

Mr. Colman, in the interesting discussion on his valuable paper previously mentioned, states that, in his opinion, sheet-piling is of no value in the reduction of head because it is not absolutely water-tight. This view is also held by irrigation engineers in India. In that country curtain-walls are made of oblong masonry under-sunk blocks, the narrow spaces between which are closed as well as practicable with concrete. Reliance, however, is not placed on these massive curtains with regard to their capability in the neutralization of uplift. In Egypt, cast-iron piles were used in the Assiut Barrage, and each joint was provided with a hollow space which was filled with cement grout.

Mr. Bligh.

It is believed, however, that the vertical length was omitted from consideration, owing to the lack of reliable data regarding their action. From these remarks, it will be evident that experimental data of a positive and exhaustive nature regarding this important subject would be a great boon to the Profession all over the world. Until such data are obtainable, it is thought that the theory of "creep", which even Philip à M. Parker,* M. Am. Soc. C. E., has termed "fascinating", should hold good as a fairly accurate and safe guide in design.

Some new developments have recently been made by the writer and published in a small work entitled, "Dams and Weirs", a copy of which has been presented to the Library of the Society.

In this the term "piezometric line" is used for the stepped line which forms the upper boundary of the area of uplift, thus distinguishing it from the hydraulic gradient, which latter is the ratio, $H : L$, L denoting the total length of travel, or creep, vertical as well as horizontal. Whatever value is assigned to vertical obstruction, it is clear that wherever it occurs a step down must result in the piezometric line and, further, as will now be shown, the latter will not necessarily be always parallel to the hydraulic gradient. Where there is no vertical curtain and no part of the superstructure is porous, the hydraulic gradient and piezometric lines will naturally coincide. A further development consists in the introduction of a filter bed of porous material, or the insertion of holes in the solid impervious floor or apron, which have the effect of stopping the uplift behind it by allowing the free exit of water. If solid material is prevented from passing out with the escaping water, all deleterious piping action is nullified; consequently, this porous length can be considered as effective as part of the line of travel of the percolating undercurrent. Thus, the direction of the piezometric line will be still further divorced from the sloping line, $\frac{H}{L}$, termed the hydraulic gradient. The introduction of a filter bed in the fore apron of the Hindia Barrage on the Euphrates was the first instance of this kind on a large scale. Another point of importance which may well be mentioned is the formation of a rear apron, impervious to water, behind a high dam, by the promotion of silt deposit by the river itself. This process can be facilitated by constructing the dam in stages, thus allowing time for the gradual deposit during several freshets. Many dams owe their safety to this complaisant action of natural forces, which otherwise, due to insufficient provision of enforced travel of the percolating undercurrent, would have failed. One further matter remains to be mentioned while there is opportunity for bringing it to notice: What is the effect, if any, of the imposition of weight on a porous foundation,

* "The Control of Water."

Mr. Bligh. subject to a head of water, in preventing or delaying its disintegration by piping?

The writer has checked the estimation of the loss of head due to vertical obstructions, according to the "creep" theory, and the results show the author's statements to be erroneous.

In Fig. 8, the length of travel is: horizontal, 745 ft., vertical $(2 \times 65 + 2 \times 85) = 300$ ft. The total value of L, then, is $745 + 300 = 1\,045$ ft. The hydraulic gradient will then be 1 in 10.45. The neutralization of head effected in the rear diaphragm would then be $\frac{2 \times 65}{10.45} = 12.4$ ft.; and that in the fore piling will be $\frac{2 \times 85}{10.45} = 16.2$ ft., as compared with the actual results, 5 and 21 ft., respectively, the discrepancy in the totals being only 2 ft. With regard to Fig. 9, the vertical component is $2 \times 125 = 250$ ft.; adding this to the horizontal, which is the same as before, that is, 745 ft., $L = 995$ ft., say, 1 000 ft., and the hydraulic gradient is 1 in 10. The loss of head due to the vertical diaphragm will then be $\frac{2 \times 125}{10} = 25$ ft. In Fig. 11, the loss of head appears to be about 20 ft. If the piezometric line, due to the theory of creep, were plotted over the experimental one, the differences would be apparent to the eye. This places the theory in a much better position than was previously apparent, and also somewhat discounts the special pleading in its favor made under a partial misapprehension of the facts. However, the experiment being made under abnormal conditions, the results cannot be accepted as convincing.

Mr. Oakes. J. C. Oakes,* M. Am. Soc. C. E. (by letter).—The application of the term "hardpan" to an uncemented material is unfortunate, as the meaning of the word in engineering literature is limited to materials hard to excavate, and more or less cemented by clay, carbonate of lime, oxide of iron, or other binding material.

In the last paragraph of "General Conditions" the stratum of so-called hardpan is referred to as "the impervious stratum." Later, in the experiments to determine the hydraulic gradient of the combined material, it is proved that this material is not impervious, but that it is readily saturated; for the porosity of the combined material is determined by that of the finer material, and, referring to the experiment to determine the gradient of the combined material, the author states:

"In the morning of the second day, fully 80% of the water was still on the surface, and at the end of 47 hours both tubes were filled with water, although the valve at the base had been closed for the entire period."

* Major, Corps of Engrs., U. S. A., Philadelphia, Pa.

Mr. Oakes.

It is apparent, therefore, that even though the valve at the bottom remained closed, the water drove out the air and fully saturated the material in the tank, as there existed no difference in head in the two tubes at the end of the test period.

The author states that it was found impossible to determine the exact hydraulic gradient of this combined material, because in another test "no water appeared in either of the tubes shortly after the valve was opened." It is evident that a sufficient length of time was not allowed to elapse, else the water would have appeared in the tubes, as proved by the saturation of this material just mentioned. He then assumes that the hydraulic gradient of the combined material "was not greater than 1:1, although it was evidently much steeper."

The writer supposes the author to mean that the slope of the gradient was not less than 1:1, although the tests apparently proved nothing. The slope of the hydraulic gradient in earthen dams is not generally as steep as 1:1, and in fact is seldom steeper than 35:100. It would seem, therefore, that careful and thorough tests should have been made before assuming 1:1 as the proper slope of the gradient for the material in question.

Furthermore, the assumption is made that, by using the combined materials in the up-stream section to secure water-tightness, and avoid transmission of water pressure through that section in any great amount, the hydraulic gradient can be made to begin at or near the up-stream toe of the dam. This is a dangerous assumption, as the material will become saturated, and some seepage will take place. Even if the fine material had been found, experimentally, to be practically impervious, it was not safe to assume that the coarse and fine materials could be mixed on a large scale in such manner that there would be, throughout the whole length of the dam, no points where the mixture would be imperfect, and where water might not seep through the combined material with greater or less facility.

Those who have had experience with filter beds, or have studied their action, know that care must be taken in placing the filter bed materials so that a uniform density will result. Whenever variation occurs, the water readily finds the paths of least resistance, and soon forms small channels through which it escapes. These channels gradually become larger, until the efficiency of the filter is destroyed. Lack of uniformity of the mixture of the materials under discussion will undoubtedly cause similar action, and, with material as fine as that described, a small quantity of seepage may cause piping, or the carrying away of the material, and the destruction of a portion of the up-stream section of the dam. Again, in Table 1, showing mechanical and chemical analyses, it is stated that 52% of the fine material is insoluble. If then, 48% of this earth is soluble, seepage

Mr. Oakes.

through it may be assisted very materially by the dissolving of the soluble parts.

With reference to the upper line of sheeting used under the first model dam, the author states: "* * * the loss of head is very small and not in proportion to the length of the sheet-piling." He does not explain why this upper row of sheeting did not accomplish as much in proportion to its length as the lower row, but, relying on what seems to be insufficient data obtained from a very small model, he abandons the upper row of piles and evolves a theory, which he claims explains the results obtained, but which, the writer thinks, does not explain those results satisfactorily. This theory is:

"* * * the sheet-piling cut-off was greatly similar in its effect to a partly closed valve, wherein the water is retarded and shows a higher pressure head just above the cut-off and a lower one just below."

He states that this theory wholly explains the ineffectiveness of the upper row of sheet-piling as a cut-off, but the writer does not see why, if the theory is correct, the upper sheeting should not also have acted as a partly closed valve. In other words, why does not the theory apply to one line of sheeting as well as to the other? It is a generally accepted principle of design that, where the apron is water-tight, the row of sheeting to be most effective must be at the up-stream edge. If the up-stream section of the dam, composed of the mixed materials, had been impervious, and the sheeting water-tight, it seems to the writer that the greatest effect, in proportion to length, would have been obtained by the upper row of sheeting. The author appears to recognize this faintly, for, on page 22, he states:

"In any dam, the correct place for a cut-off wall, under conditions such as these, namely, a pervious foundation, is at, or near the up-stream toe."

This statement is certainly correct when the dam is impervious, but it is not correct in case of a pervious dam or apron. It seems to the writer that the inefficiency of the upper row of sheeting can be readily explained, either by its lack of water-tightness, or by seepage through the material of the dam, between the pool and some point or points of the foundation down stream from the line of sheeting. Owing to the short period of time during which the experiment was carried on, the material above the second line of sheeting, being much thicker, did not become thoroughly saturated, seepage did not proceed directly from the pool through the material of the dam to a point down stream from the second row of piling, and consequently this row of piling did accomplish what was to be expected of it.

Although nothing is said about the length of time to which the model was subjected to the various heads of water, it is noted that, on each

of the days that the work was carried on, from four to eight experiments were made. In one case five experiments were made in the afternoon, so that it is very evident that a very short period of time was allowed for each experiment. With the material as fine as that described, complete saturation near the core-wall could not have been expected in such short time, but if, in each experiment, the head had been maintaned for a number of days, complete saturation would have taken place, and it would then have been found that the lower line of sheeting was also inefficient. Mr. Oakes.

Nothing is stated in the paper as to the percentage of voids in the coarser material, but the impression is given that water passed through this material very freely. If, then, after a time, the material in the upper section of the dam becomes thoroughly saturated, the writer does not see what is to prevent the carrying away of the fine material of the dam section through the coarser material of the foundation, thereby causing ultimate failure of the dam. He can see no advantage whatever in placing the line of sheeting as shown in Fig. 9. If the material forming the upper section of the dam is impervious, then the line of sheeting should be near the up-stream edge. If the material is not impervious, then it should be under the core-wall and connected therewith to form an impervious sheeting from crest of dam to foot of piles.

Fig. 8 indicates that the proposed dam is to withstand a depth of water of 100 ft., and Fig. 9 shows the design with a base approximately 700 ft. wide. For 100 ft. of this width the dam section is from 10 to 15 ft. thick. As the core-wall and sheet-piling do not necessarily increase the line of percolation, the percolation factor, or ratio of length of line of percolation to head, is approximately 7, which is much too small, if the writer understands the nature of the materials of foundation and dam section. Furthermore, it may be possible for seepage to take place along lines at 45° with the vertical and meeting at the foot of the core-wall, in which case this factor becomes about 2¼. This possibility is increased because the core-wall will tend to concentrate the seepage at its foot, and such seepage will escape under the wall and upward through the down-stream section, probably at an angle of approximately 45 degrees.

According to the best authorities, these factors are too low. Among these authorities may be cited Mr. W. G. Bligh, whose experience and theories are dismissed by the author with the statement:

"This theory gives structures of ample dimensions, as has been shown by practice, but was found to be incorrect, after due experimenting, as will be noted later."

If the author means that the experiments that he describes in this paper prove the falsity of Mr. Bligh's theory, the writer must dis-

Mr. Oakes.

agree with him, as, in his opinion, the model was too small, the experiments too few, and too little time was devoted to each experiment to warrant any such broad statement. It seems to the writer that erroneous conclusions, or at least unwarranted conclusions, were drawn by the author. It seems very doubtful whether results obtained with such a small model may be considered to hold for heads 120 times as great as those used in the experiments. In writing about the failure of a dam on sand foundations, and giving his opinion on the values of required percolation factors, Mr. Bligh has sounded a warning which exactly covers this case, as follows:

"The proper value of this factor is found, not by artificial experiments on a small scale, but by undisputable statistics which bear on the capacity of the various pervious materials which may compose a river bed to resist the undermining influence of the pressure of the water upheld."

A remarkable statement occurs on page 15, as follows:

"The long up-stream slope was given in order to allow the downward pressure of the water over the up-stream section to have a balancing effect on the upward pressure beneath the dam, as blow-outs would be improbable in this portion of the dam."

If the writer understands this sentence correctly, the author seems to feel that, although improbable, blow-outs may occur in the bottom of the reservoir, and he extends the dam up stream to prevent this by providing something for the down-stream pressure of the water to act on.

Although the author, under "Conclusions", has mentioned the principal causes of failure of earthen dams, he makes a number of statements in the last two paragraphs that do not seem to follow necessarily from the experiments described in this paper. Although possibly true, he has certainly not proved that:

"Where the sand or gravel foundation is of very great depth, a short cut-off would not be as efficient as in a shallower foundation, * * *."

Nor has he proved by his design that:

"* * * where models are constructed, results can be obtained which enable the engineer to make correct designs better than by any other method of study."

It appears to the writer that the use of a very small model, and the lack of certain precautions, such as allowing sufficient time in each experiment for the material to be thoroughly saturated, and the assumption that the material is impervious, or practically so, where it is very evidently not so, has led the author to arrive at erroneous conclusions, and to design a dam, which, if constructed as designed, is likely to fail.

Mr. Oakes.

The writer has had occasion to be deeply interested in the subject of dams on sand foundations, and has found no example of a dam on a pervious foundation constructed with a cross-section approaching in boldness that designed by the author. It will be very interesting, indeed, to know whether the dam has been constructed as designed, and, if so, to learn at a later date how it fulfills its purpose.

There is a statement at the end of the paper to which the writer is willing to subscribe in its entirety, as follows:

"Each design is a problem in itself, and the experiments described herein should not be misinterpreted, or applied too broadly."

Mr. Grunsky.

C. E. GRUNSKY,* M. AM. SOC. C. E. (by letter).—The question of the effect of water-tight sheet-piling which does not extend entirely through a layer of pervious material, on the hydraulic gradient in that layer, is one of vital importance in many cases where either a dam must be built on a pervious foundation or not built at all. Every contribution toward the solution of this problem should be welcomed by the Profession. The writer suggests one which should be tried out:

Let D represent the depth of the gravel bed, in feet.

Let d represent the depth of the sheet-piling, in feet.

Let s represent the fall, in unity, of the hydraulic grade line for those portions of the gravel bed not affected by the sheet-piling. This is the gradient that will cause a flow in the gravel equal in quantity to the leakage under the dam.

Let S represent the fall in unity required to create the velocity with which the water flows under the sheeting through a section of gravel from B to C, Fig. 12, being a layer with a thickness of $D - d$ feet in which the water is assumed to flow horizontally.

Let h represent the reduction of hydrostatic pressure due to the sheet-piling, expressed in feet.

The velocity of the water in the gravel is proportional to the hydraulic gradient. Therefore,

$$S = \frac{D}{D - d} s \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

The velocity at which water moves through the gravel toward the gravel-filled space, O-B, under the sheet-piling, Fig. 12, decreases with the distance from O-B. Uniformity of velocity at uniform distance from O-B may be assumed. Up stream from the sheet-piling, therefore, the hydraulic gradient should be practically constant at each point of each of the full concentric lines shown in the diagram, and there should be a gradual increase of the gradient from s at the outermost line passing through A, C, and D, to the gradient, S, at all points on the line, O-B. Down stream from the sheet-piling,

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Mr. Grunsky.

on the other hand, there should be a gradual decrease of the hydraulic gradient from S at the line, O-B, to s at the outermost of the concentric lines passing through A, C', and D'.

The change in the gradient from s to S, up stream from the line of sheeting, and from S to s, down stream from the same line, is practically proportional to the distance from O-B, because the increase of the area of the gravel through which the water must pass is proportional or nearly so to this distance, and velocity and therefore velocity head in a gravel bed are inversely proportional to cross-sectional areas.

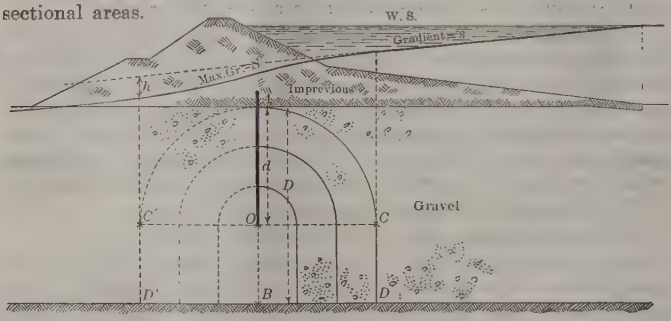

FIG. 12.

It follows that the total fall of the hydraulic grade line throughout the distance, C-C', will be

$$\frac{S+s}{2} \times (C\text{-}C').$$

But $C\text{-}C' = 2\ (A\text{-}O) = 2\ d$.

The total fall of the hydraulic grade line in the distance, C-$C' = D$-D', is, therefore,

$$\frac{S+s}{2}\ (2\ d), \text{ or } d\left(\frac{S+s}{2}\right).$$

The fall of the grade line in the same distance, if there were no sheeting, would be $2\ d\ s$.

Consequently:

$$h = d\ (S+s) - 2\ d\ s \quad \ldots\ldots(2)$$

$$h = d\ (S-s) \quad \ldots\ldots(3)$$

Substituting the value of S from Equation (1),

$$h = d\left(\frac{D}{D-d}\,S - s\right) \quad \ldots\ldots(4)$$

$$h = \frac{d^2\ s}{D-d} \quad \ldots\ldots(5)$$

or, $$h = \frac{d\,s}{D - d} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (6)$$ Mr. Grunsky.

When there is no sheeting, $d = 0$ and $h = 0$. When the sheeting extends to bed-rock, then $s = 0$, $d = D$, and h is indeterminate; the flow is completely checked. This also appears from Equation (1), which makes $S = \infty$. The fall in unity at the sheeting is infinity, and the hydraulic gradient comes to a full stop at the line of the sheeting.

Equation (5), applied to the F. C. Horn model, for the particular case when $s = 1:9$, indicates the following for 85-ft. sheeting:

For $D = 240$ ft.
and $d = 85$
and $s = 1:9$

$$h = \frac{85 \times 85}{155} \times \frac{1}{9} = 5.2 \text{ ft.}$$

For 40-ft. sheeting $d = 40$,

and $$h = \frac{40 \times 40}{200} \times \frac{1}{9} = 0.9 \text{ ft.}$$

For 125-ft. sheeting, the loss of head would be 15 ft.

These results, it is believed, are fairly dependable. The hypothesis on which they are based is certainly more reasonable than the "creep theory" for which the writer has found no theoretical justification.

It will be noted that the examples here given apply only to the particular cases in which $s = 1:9$, that is, $s = 0.11$. To make comparisons with the experimental results obtained with the model, the first determination should be of the value of s, which will vary for each length of sheeting. The effect of the sheeting expressed by the total drop of the gradient is proportional to s, which is a variable.

The slope of the hydraulic gradient, or value of s, to be used in the equation for h can be ascertained for a model from the volume of flow through the gravel, by making experiments with varying quantities of water, and a corresponding variation of head, before the sheeting is placed in position. For this experiment, however, the gravel should be covered with an impervious material, so that the entire discharge must pass through the gravel. After the sheeting has been placed and the flow of water under it has been ascertained, this volume will determine, from the results of the preceding experiments, the proper value of s.

There is now to be considered the problem of determining the effect of sheet-piling when the value of s which will obtain after the sheet-piling is in place is not known.

Mr. Grunsky.

Without any sheeting under a dam of impervious material the hydraulic grade line may have any of the positions, *a*, *b*, or *c*, Fig. 13. When in the position indicated by the line, *a*, the outfall of the water into a channel or other receiver below the dam takes place with a more or less abrupt release of pressure, the dam is in danger. When the hydraulic grade line is in the position, *c*, the flow in the gravel under the dam is so small that water will not appear at the down-stream toe of the dam. When in the position, *b*, all water appearing at the down-stream toe will be under no pressure at the ground's surface, and the water passing under the dam ceases to be a menace.

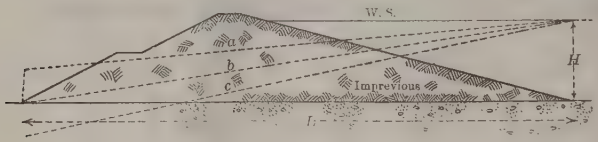

FIG. 13.

Let it be assumed that, in the case of the finished structure, which has the benefit of sheet-piling, the hydraulic grade line passes through the down-stream toe, so that the average gradient of the line is $H \div L$. For this particular case, the effect of the sheet-piling can be estimated as follows:

The average gradient throughout a distance of about d feet on either side of the sheeting, as already shown, will be $\frac{S+s}{2}$.

The value of H will be

$$H = (L - 2\,d)\,s + h \quad \ldots\ldots (7)$$

$$H = (L - 2\,d)\,s + \frac{d^2\,s}{D - d} \quad \ldots\ldots (8)$$

from which

$$s = \frac{H}{L - 2\,d + \frac{d^2}{D - d}} \quad \ldots\ldots (9)$$

$$s = \frac{H\,(D - d)}{L\,(D - d) - d\,(2\,D - 3\,d)} \quad \ldots\ldots (10)$$

This value of s substituted in Equation (5) will make

$$h = \frac{d^2}{D - d} \times \frac{H\,(D - d)}{L\,(D - d) - d\,(2\,D - 3\,d)} \quad \ldots\ldots (11)$$

$$h = \frac{d^2\,H}{L\,(D - d) - d\,(2\,D - 3\,d)} \quad \ldots\ldots (12)$$

Mr. Grunsky.

This expression for h represents the effect of the sheet-piling when the hydraulic grade line has been brought into the position, b, Fig. 13. In the case of earth dams, it will represent the benefit, expressed in head, which may be expected from the sheet-piling under a dam at the safety margin, and therefore the maximum benefit that may be hoped for from the use of sheeting.

The experiments made with the model are not described in the paper so that they enable one to test the equation by the tabulated experimental results. The low heads recorded for the points, H to K, indicate that the water was allowed to flow out of the drain cock. This would cause the gravel bed to act as a filter, and would produce a condition at variance with that which prevails at the dam site, where a free escape of water at, but not below, the level of the stream bed may be assumed. It is possible that the readings on the upper line of tubes and on the lower were not simultaneous, but, if this is the case, this should have been explained in the paper.

Mr. Pease.

H. T. Pease,* Assoc. M. Am. Soc. C. E.† (by letter).—This paper is an interesting example of the method expressed by the phrase *reductio ad absurdum*. It shows how far a line of reasoning may be carried beyond a logical limit.

No engineer would think of designing a bridge 100 ft. long using a test of a bridge 10 in. long as a basis for the design; especially would he consider the proceeding irrational if the conditions under which the model was tested were different from those actually to be met by the design. Mr. Hays' procedure is analogous to such a case. He has built a very large edifice on a very narrow base, a pyramid standing on its apex. Designing a dam 100 ft. high on the result of the tests of a model 10 in. high is a good deal like designing an ocean liner on the lines of a canoe.

The author's design would undoubtedly prove safe for a dam several feet in height, but that it would prove safe for more than 100 ft. of height is improbable; certainly, he does not prove his case. In a dam 10 in. high the effects of capillarity and viscosity of the water would be largely felt. In fact, they would undoubtedly comprise a large percentage of the total forces acting in this case. Such, however, would not be the case with the 100-ft. dam. These forces here would be negligible.

The final design shown in Fig. 9 indicates a slope of 2:1 on the up-stream face and a slope of 1½:1 on the down-stream face to a depth of 60 ft. Considering the nature of the material, as it is described in the paper, in fact, considering almost any material of which an earth dam might be constructed, this should be regarded

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† Now M. Am. Soc. C. E.

Mr. Pease. as an unsafe section for a dam of this height. A slope at least lighter than 1½:1 should be used for the down-stream face.

The core-wall shown reaching to the top of the dam, in fact, the core-wall in any earth dam, is in most cases a "delusion and a snare". If the material of the up-stream portion is tight enough to hold water, the core-wall is unnecessary; whereas, if it is not tight enough to hold water, the up-stream portion serves no purpose whatever, except as a support for the core-wall when the dam is empty. Moreover, in such a case the pressure of the entire head, supplemented in many instances by an added weight of earth in a semi-fluid state, is brought to bear on the section at the core-wall. To say the least, such a stress would be hard on a thin wall such as is shown in the diagram, because the material supporting it is not rigid.

The up-stream toe would seem to be drawn out to too thin a section. If the material composing it is at all pervious, what is to prevent the finer particles from being carried down into the coarse substratum until this toe becomes as pervious as the ground beneath?

The author states that he has proved the "line of creep" theory incorrect, "after due experimenting". His experiments consist of the test of a model constructed on a very small scale, but of the exact proportions of the actual dam. Every detail but one is thus proportioned; while he was at it he should have reduced the sizes of the grains of the material of which the model was composed to the same scale, *i. e.*, 1:120. A length of 24 in. of the natural sand and gravel placed under the model represents the 240 ft. of the same material which will be found at the dam site, and in this sheet-piling is placed reaching to within 11½ in. of the floor of the water-tight box in which the model is built. This is to represent piling which will be driven to a depth of 125 ft. below the base of the dam or within a distance of 115 ft. of an assumed rock surface below. That the actual subsurface conditions are thus duplicated in the model is hard to believe.

After making some tests for loss of head, the author concludes that the sheet-piling reaching to within 11½ in. of the floor of the box acts as a partly closed valve. There would appear to be an element of truth in his contention—in this case.

The conclusions to be drawn from the paper are:

That the author has shown a commendable enthusiasm and an attention to detail often lacking in such an investigation, but that he has over-estimated the usefulness of the data collected;

That the test conditions do not duplicate the actual conditions to be encountered; and

That the safety of his design is open to suspicion.

MALCOLM ELLIOTT,* ASSOC. M. AM. SOC. C. E.† (by letter).—The author of this interesting paper deserves credit and the thanks of engineers for efforts to throw light on a subject about which little is known, and for making his results and conclusions available to others. In commenting on this series of experiments, the writer wishes to disclaim any intention of merely picking flaws. What appear to be inconsistencies and errors may be due to misunderstandings which should be cleared up by other discussions. Mr. Elliott.

The problem of determining the upward pressure on the base of a dam built on a pervious foundation may be likened to that of determining the lateral pressure of earth on the back of a retaining wall. No general theory of earth pressures will take care of all possible conditions as to cohesion and friction; neither will any general formula represent truly all possible degrees of porosity and permeability in the case of upward pressure on a dam. Nevertheless, for earth pressures, we are content to go on using the Rankine formula, well knowing that it does not accurately represent the true pressures, but using it because it gives practical values sufficiently near the truth for safety in design, when a reasonable margin of safety is provided. In like manner, the so-called "line of creep" theory may not accurately determine the upward pressures at the base of a dam, but it can be depended on as a generally safe rule for design.

The author recognizes that his results are not generally applicable; in his closing words he says that each design is a problem in itself, and that the experiments should not be applied too broadly. He might have added that his results could be used only for the dam which he was designing, and even then with extreme caution and proper allowances because of the small scale on which the experiments were made.

In using a model for any kind of a demonstration, it is desirable, of course, to reproduce, as faithfully as possible, the actual structure for which the strength or behavior is to be determined. This ideal cannot be attained in constructing models of dams on such a small scale as that used by the author.

In his model, the dam and sheet-piles are very much reduced in scale, though the soil particles are of the same size as those used in the full-scale dam, and the water is of the same degree of fluidity. The interstices in the sand, where it is in contact with the sheet-piles, are much larger, in proportion to the length of the piles, than they are in the actual dam. If the soil were reduced to the same scale as the dam and sheet-piles used in this series of experiments, it would be an impalpable powder; or, looking at it in another way, if the soil particles used in the model were enlarged so as to have the same relation in size to the actual dam as they have to the model dam, the

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Mr. Elliott.

underlying material (Curve *A*, Plate IV) would be a mass of stones and boulders, 90% of which would be composed of pieces more than 2 in. in diameter, 80% would be stones more than 12 in. in diameter, and the largest "particles" would be boulders 10 ft. in diameter.

The tendency of the water to "creep" along the surface of the sheet-piles would possibly be greater in the actual dam than in the model, because of this difference in the size of the particles and their interstices when compared with the length of the sheet-piling. This seems to be substantiated by the greater proportionate loss of head at the second or longer row of sheet-piling. Here the measured loss of head for a 100-ft. head was from 19 to 21 ft., and the computed loss of head, according to the "line of creep" theory, with the hydraulic gradient of 1:9, would be *18.9 ft. From the very small loss of head at the first or shorter row of sheet-piling, the author deduces that this piling is of little value; and, in his second model, he eliminates it and adds to the length of the second row. The writer believes that if the author had used a model on a larger scale, or if he had increased the length of the up-stream row of piles in the model which he used, the loss of head due to the piles would have been more nearly in accord with that computed with the "line of creep" theory.

The small loss of head at the up-stream piles might also be explained in part by leakage through the apron on the up-stream slope of the dam. If, as is claimed by the author, this covering were impermeable, the logical place for the single row of sheet-piling would be in the up-stream trench, instead of the down-stream trench, and the concrete core-wall would be superfluous. Even a small leakage through this up-stream apron over the top of the upper row of piling would raise the hydraulic grade line down stream from the piles, thus apparently reducing the loss of head due to the piles. In his paragraph immediately preceding his conclusions, the author recognizes the possibility of leakage over the tops of the piles, when he says the piling was placed "sufficiently far back to prevent any water from seeping through the dam to the down-stream side of the piling", yet he does not attribute to this possibility the apparent failure of the "line of creep" theory at the upper row of piles.

One of the striking characteristics of the author's experiments is the apparently large loss of head where the water enters the sand. Thus, if the curve for Experiment 9, Fig. 10 (*a*), is extended to the toe of the dam on the same slope as between Tubes *B* and *C*, it appears that there will be a loss of head at entry of about 5½ in., out of a total head of 10 in. In giving his results in feet (the actual readings multiplied by 120), the author would apparently have us believe that

* Not 9.4 ft., as stated by the author. This is the loss of head on only one side of the sheet-piling.

Mr. Elliott.

the values he gives would approximate those to be expected with the full-scale dam, and that the loss of head at entry would be about 55 ft., out of a total head of 100 ft. If this were true we would have a surprising departure from hitherto accepted theories, which have been to the effect that the entry head, though appreciable, is not generally a large proportion of the total head.

It is seen immediately, however, that accepted theories as to the amount of the entry head are not destroyed by the results of these experiments. The loss of head at entry is more nearly an absolute than a relative quantity; it does not vary with the total head, but with the velocity head. The velocity head with the actual dam will certainly not be 120 times that with the model dam; therefore the loss at entry will be relatively much smaller than with the model dam, leaving a much larger proportion of the total head to be overcome by friction, and thereby changing materially the slope of the hydraulic grade line.

Yet, if we should accept these experiments as conclusive, even with the limited applicability which the author claims for them, they would indicate a loss of head at entry, in the actual dam, of 55 ft., though, as a fact, it will be nearer 5½ in.

This difference between the model and the actual conditions is one of the fundamental reasons why the solution of this problem should not be attempted on such a small scale. The model should be of such size, compared with the actual structure, as to make negligible the error introduced by assuming that the loss of head at entry is proportional to the total head.

The departure of the author's results from those obtained by the "line of creep" rule may be explained largely by the relatively great loss of head at entry which is due to the small scale on which the experiments were made. With experiments on a larger scale, or under actual conditions, the loss at entry is insignificant, compared with the total head, so small, in fact, that it is entirely neglected in the "line of creep" theory as ordinarily applied.

The writer confesses not being able to understand the author's reference to the "hydraulic gradient of the underlying material" which he assumes to be 1:9 throughout. Apparently, he determined this by his preliminary experiments with the circular tank, and then for his trial design assumed that a certain down-stream slope of the dam would be necessary to "hold up the 100-ft. head". As the writer understands experiments of this kind, the purpose should be to determine what hydraulic gradient is required in order to reduce the velocity of the percolating water to such an extent that it will no longer have the power to transport the particles composing the medium through which it percolates. If the circular tank experiments gave

Mr. Elliott.

any information on this point, the author has not stated it in his paper. Clearly, the "hydraulic gradient of the material" could have been varied at will, either by increasing the head or the area of the outlet at the bottom of the tank, without in any way changing the character of the filling.

The author then assumes in Fig. 8 that the hydraulic gradient under the dam, according to the "line of creep" theory, will be 1:9, notwithstanding that this assumption makes the hydraulic grade line intersect the base of the dam well within the toe. The computed hydraulic grade line should intersect the base at the down-stream toe, to be consistent with the "line of creep" theory which the author wished to represent for comparison with the actual tube readings. According to the "line of creep" theory, and the dimensions adopted for the dam, the hydraulic gradient for a 100-ft. head should have been about 1:10.5 instead of 1:9.

Notwithstanding the foregoing inconsistencies, the author seems to have arrived at a generally safe and economical design for the dam which he had under consideration. Placing the relatively impermeable material on the up-stream face should result in making the hydraulic grade line begin near the toe instead of near the crest as it would have done if the dam had been built entirely of the material with which it was started.

The location of the piling near the center instead of near the up-stream toe is justified if the up-stream slope is not sufficiently tight to exclude the water. With a masonry or other water-tight dam, the piling, of course, should be near the toe.

The combination of local materials, in order to secure maximum density, is noteworthy, and the application of sieve analysis curves to this problem seems to be original. It would be interesting to know the construction methods by which the intimate mixture of the two materials was obtained.

Mr. Wegmann.

EDWARD WEGMANN,* M. AM. SOC. C. E.—The author is to be congratulated on the scientific manner in which he has worked up the results obtained with his model of an earth dam. Although the paper is interesting in this respect, the speaker does not think that any general conclusions, drawn from the results obtained with such a small model, will be applicable to the construction of a high earth dam.

In reference to Fig. 9, it is the speaker's opinion that, although in the experiment the water passed along the sheet-pile cut-off to its bottom and then rose again, in a high dam, built according to the model, the water would be very apt to take a short cut and pass over the top, instead of under the bottom, of the piling.

* New York City.

Mr. La Rue.

E. C. La Rue,* Assoc. M. Am. Soc. C. E. (by letter).—This paper will be of exceptional interest, for only meager data are available relating to the proper design of an earth dam having a gravel foundation. The partly completed dam, described by Mr. Hays, formed a part of an irrigation project on which the writer has prepared a water supply report. He has also examined the dam site, and is therefore specially interested in the paper.

In some parts of it the author has not made clear the procedure followed in his experimental work. For example, in determining the hydraulic gradient of the material used in the original dam and that to be used in the upper portion of the actual structure, the details of the methods are not made clear. The writer is in doubt as to the following points:

1.—When the readings were taken, was the quantity of water leaving the same as that entering the tank?

2.—Before the soil was placed in the tank, were tests made in order to determine the head required to force the water through the valve? If the coarse materials were tested to determine the hydraulic gradient, then, with the same quantity of water flowing into the empty tank as that used in the test, the head required to force this water through the valve could be determined. If, during such tests, it was found that the water level in the tank was above the intake to the lower glass tube, then the author's results are in error. During the tests, with the soil in the tank, the head required to force the water through the valve would be effective to a higher level in the tank. It is probable that the water level in the lower tube was not affected in the manner referred to, but the author has not made this point clear.

On page 5, in referring to the North Dike of the Wachusett Dam, Mr. Hays states:

"A flat hydraulic gradient, caused by the water in the reservoir seeping under the dam, called for a large quantity of material, in order to withstand the upward pressure under the down-stream portion of the dam."

In referring to the Gatun Dam, he states:

"A flat down-stream slope causes the percolating water to travel a long distance before a free opening is encountered, thus causing the upward pressure to be consumed by friction."

As the down-stream sections of these dams are constructed of pervious material, the water cannot exert an upward pressure on their bases. Probably the author means that the flat hydraulic gradient makes it necessary to construct the down-stream section of the dam with a flat slope in order to prevent the line of saturation from intersecting the down-stream face of the dam above the toe.

* Salt Lake City, Utah.

Mr. La Rue.

On page 15, in referring to the hydraulic gradient for the selected material to be used in the upper section of the actual structure, the author says: "From this it was assumed that the hydraulic gradient was not greater than 1:1, although it was evidently much steeper." This statement is not consistent. It would appear that he meant to say that the hydraulic gradient was not less than 1:1, and it was evidently much steeper.

On the same page, in referring to the model dam, the author says:

"The long up-stream slope was given in order to allow the downward pressure of the water over the up-stream section to have a balancing effect on the upward pressure beneath the dam, as blow-outs would be improbable in this portion of the dam."

Although the down-stream pressure of the water over the up-stream section does have a balancing effect on the upward pressure beneath the dam, this surely was not the reason for adopting a "long up-stream slope". The author assumes that the material in the up-stream section will be practically water-tight. It would appear, therefore, that the flat up-stream slope was adopted in order to force the hydraulic gradient to begin farther up stream.

The author concludes that, with 10 in. of water in the reservoir above the model dam, the head is entirely consumed before the water reaches the lower toe of the dam. Using Test No. 17, Table 3, the writer has drawn lines of equal pressure, and from these produced the lines representing approximately the direction of flow through the foundation of the model dam. The direction of flow was downward in that portion of the foundation above the sheet-piling. Below the sheet-piling the flow lines rose slightly and then appeared to turn in the direction of the drain valve, shown in Fig. 8. If the pressure had been observed at various points in the foundation below the lower toe, there is no doubt that it would have shown conclusively that the direction of flow was toward the drain valve. Mr. Hays has shown the hydraulic gradient for the material used in the foundation to be about 1:9. With a free escape for the water, as is provided by the drain, little, if any, pressure would be observed immediately above the drain valve. Extending the hydraulic gradient back from the drain valve with a slope of 1:9, it will be seen to intersect the line representing the base of the dam at a point 2 160 ft. above the drain, or 1 040 ft. up stream from the upper toe of the actual dam. All the pressure observations taken by the author must have been affected by the drain. Under these conditions, the water could not rise to the base of the dam near the lower toe. In the model, the author has imposed conditions which would not be reproduced in actual practice. If the dam is to operate under the same conditions as the model, then 360 ft. below the lower toe of the dam a trench, 240 ft. deep and 40 ft.

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wide, must be constructed across the canyon. This trench must be filled with large stone, in order to provide a perfect drain. In the bottom of the trench, or at the 240-ft. level, there must be openings with sufficient capacity to carry off the water as it arrives. The water must then be conveyed to a reservoir of infinite capacity. In the model, the drain should be 216 in. from the lower toe, instead of 36 in., or it will affect the water pressure under the base of the dam. If the writer's contentions are correct, the results of the author's experiments are of little practical value.

That the drain referred to has affected seriously the results of the experiments is indicated by the excessive seepage through the foundation of the model. In Test No. 17, Table 3, the seepage, per linear foot of the model, was 0.00222 cu. ft. per sec. The slope of the line of saturation in the model is assumed to be the same as that of the actual dam. The length of travel in the model is proportional to that of the actual dam. The head and all other dimensions in the model being proportional to those of the actual dam, the seepage through the latter, per linear foot, will be 120 times that through the model per linear foot. The length of the actual dam is to be 2 000 ft., and its left end will extend into a bank of gravel similar to that which is to compose the foundation. Considerable excavation will be necessary in order that the water-tight section of the dam can be carried well into this bank. It is assumed that the area exposed to water pressure will be equivalent to the area of 1 200 ft. of maximum section. The seepage through the actual dam, therefore, would be $120 \times 1\,200 \times 0.00222$, which equals 320 cu. ft. per sec.

Mr. Hays has not disproved the "line of creep" theory. In the first test, with the two rows of sheet-piling, he found a small loss of head at the upper row of piling. This, no doubt, was due to water passing through the upper section of the dam and entering the foundation both above and below the upper row of sheet-piling.

The pressure at *D*, Fig. 8, below the lower row of sheet-piling, was no doubt due to the water from the upper section of the dam entering the foundation between the piling and the cut-off wall. In fact, the drain would prevent the water from rising to *D* after passing below the lower row of sheet-piling. If Mr. Hays will place the drain 216 in. below the lower toe of the model dam, separate the upper section of the dam from the foundation, with a sheet of tin, and connect the tin with the cut-off wall and the two rows of sheet-piling with water-tight joints, then the hydraulic gradient will begin at the upper toe. Under these conditions, the effect of the two rows of sheet-piling can be determined, and undoubtedly the "line of creep" theory, somewhat modified, will prove to be correct. That is, instead of following down one side of the sheet-piling and up the other, thence along the base of

Mr. La Rue.

the dam to the second row of sheet-piling, etc., the water will follow down the upper side of the first row of sheet-piling and thence in the general direction of the lower end of the second row of sheet-piling.

Assuming that there were no other defects in the model, the writer believes the results of the tests to be unreliable for the following reasons:

1.—The model dam, being only 11 in. high, and subjected to a head of 10 in. of water, the entrance head and capillary action would, no doubt, affect the hydraulic gradient to such an extent that the pressure observations in the model would not indicate the action of the water in the final structure, where similar material will be subjected to a head 120 times greater than that on the model.

2.—The model dam was constructed of the same material as that to be used in the actual structure. It would seem that the material for the former should have been coarser than that to be used in the latter. The ratio between the coarseness of the materials in the two structures, which would result in the action of the water on the model being comparable to that on the actual dam, could perhaps be determined by extensive experiments.

The writer makes the following suggestions:

1.—That the author construct a model which will represent exactly the original dam as it was to have been built. It is probable that the results of tests on this model would show the structure to be safe, provided the model were constructed with the drain 36 in. below the lower toe. As a matter of fact, with a head of 24 ft. on the actual dam, the water passed freely through the partly completed structure.

2.—That the author could obtain more reliable information from a series of tests on models 1, 2, 4, 8, and 12 ft. in height, provided the drain shown in Fig. 8 were placed at a section 216 in. down stream from the lower toes of the respective dams. From the results of the experiments with each of these models, it is possible that some sort of a curve could be prepared which might be extended to show approximately the action of the water on the final structure under a head of 100 ft.

Conclusions.—

1.—The author has not stated clearly how each experiment was carried on. In some places it is necessary for the reader to assume that the experiment was conducted under certain imposed conditions.

2.—The model dam used by the author was too small to give reliable results.

3.—The pressure, at all points observed in the tests, was affected by the drain; therefore, the results of the experiments are of little practical value.

4.—The drain in the model should have been placed 216 in. below the lower toe, instead of 36 in. Mr. La Rue.

5.—The author has not disproved the "line of creep" theory.

6.—It would appear that too much dependence was placed on the impermeability of the material in the upper section of the dam.

7.—If the dam, as designed, were constructed to operate under the conditions imposed in the model, the flow under it would be about 320 cu. ft. per sec.

8.—It is the writer's opinion that the sheet-piling should be placed beneath the core-wall. If the core-wall in the partly completed dam is to be used, then the sheet-piling should be placed near the core-wall on the up-stream side. A water-tight connection should be made between the core-wall and the sheet-piling. The writer, being somewhat familiar with the conditions at the dam site, feels that, at best, there will be considerable seepage under the dam, and for this reason he would suggest that the slope of the down-stream face, below the 30-ft. berm, be made as flat as $3\frac{1}{2}:1$ or $4:1$. The proper slope for the down-stream face could no doubt be determined by extensive tests on a series of properly designed models of various heights.

Mr. Bacon.

GEORGE M. BACON,* M. AM. SOC. C. E. (by letter).—The main objection to drawing conclusions from these experiments seems to be the assumption that the foundation as composed for the model represents sub-surface conditions at the dam site, an assumption hardly correct. What part of the foundation in the model corresponds to the actual condition on the ground, which allowed sheet-piling to sink "as deep as 32 ft. with one or two blows from a 1 700-lb. hammer"? The mountain streams formed "great cones, or fans, of very porous material". Was this material duplicated in the model, and, if so, where? In an experiment of this kind, it is vital to duplicate the actual conditions which are the subject of investigation. No ingenuity of observation and recording can minimize the importance of this. There is practically nothing in the paper showing how the foundation in the model was formed, or indicating its similarity to actual conditions.

The author's theoretical analyses are interesting, but, should they serve as a basis for the solution of the problem actually presented? If the premises are not correct, any deductions from experiment are not only of no value, but can easily be harmful as well as misleading.

Mr. Petterson.

H. A. PETTERSON,† ASSOC. M. AM. SOC. C. E. (by letter).—The writer is a great believer in experimental engineering, and realizes that much of the advance made in engineering knowledge is due to the researches of careful and ingenious experimenters. He cannot believe, however, that experiments made on models with a depth of

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† Tientsin, China.

Mr. Petterson.

water of only 10 in., will bring forth results of any great practical value in designing a dam to impound water 100 ft. and more in depth. Our knowledge of the underground flow of water is not as complete as it ought to be. This is true especially of underground flow as affected by cut-off walls penetrating only part way into the porous stratum.

The most reliable experiments, however, would be those made on existing dams and weirs built on porous foundations. There are many structures in different parts of the world on which experiments could be made, and these could follow essentially the methods developed by Mr. C. S. Slichter.* Until such experiments are made, the writer, for one, would advocate following present methods, a brief presentation of the underlying principles of which will be given.

The principles governing the design of an earth dam with impervious core-wall to impervious foundation need not be reviewed, as they are treated in any number of good textbooks. The problem of securing water-tightness in earth dams is essentially one of securing the maximum density of the material; and the laws governing this are known by the Engineering Profession, even though not universally applied. Strict adherence to these principles in earth dam construction involves extra cost, which is not always warranted by the results obtained.

This discussion, therefore, will be confined to the principles involved in the design of an earth dam on a porous foundation of such great depth that an impervious cut-off wall to an impervious stratum is financially impracticable. It will be assumed that the dam will be made relatively impervious, and safe against ordinary methods of failure. The principles to be elucidated are the securing of stability against the possible destructive effect of water flowing under the dam (not through it); also, as it is assumed that the dam is to impound water in a reservoir, the investigation of the quantity of water lost by percolation is important from an economic standpoint, though it may in no way affect the stability of the dam.

A rational design cannot be made without a comprehensive grasp of the laws governing the flow of underground water. A very brief summary of present knowledge on these laws will be given, for the purpose of calling attention to the incompleteness of that knowledge and to make clearer the writer's comments on certain of the author's statements.

Laws of Underground Flow.

Hazen's formula, reduced to English units,† is

$$Q = 3.28\, c\, d^2 \frac{h}{L} \frac{A}{60} \frac{(t+10)}{60} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

* Described in Water Supply Papers, Nos. 67 and 140.

† Report, Mass. State Board of Health, 1892, p. 553; also, Turneaure and Russel, "Public Water Supplies", p. 96.

Mr. Petterson.

Slichter's formula* is

$$Q = \frac{16\,272\,d^2}{k}\,\frac{h}{L}\,A\,[1 + 0.0187\,(t - 32)] \quad \ldots\ldots\ldots (2)$$

Baldwin-Wiseman's formula† is

$$v = c_1 c_2 \frac{h}{L} \quad \ldots\ldots\ldots (3)$$

In these formulas, c, c_1, c_2, and k, are coefficients. The values of c in Hazen's formula vary from 400 to 1 000. The values of k are tabulated by Slichter, and vary only with porosity. c_1, in Equation (3), is proportional to cd^2 and $\frac{d^2}{k}$ in Equations (1) and (2). c_2 corresponds to the temperature correction of Equations (1) and (2).

Q = discharge, in cubic feet per day, through the area, A;
A = area of cross-section, in square feet, normal to the line of flow;
h = difference of water surface, in feet, for two points distant L feet apart;
L = distance, in feet, measured in direction of line of flow;
d = effective size of sand grains, in millimeters, determined by mechanical analysis with sieves in Hazen's formula, and by the use of King's aspirator‡ in Slichter's formula;
v = velocity of percolation;
t = temperature of the water, in degrees, Fahrenheit.

These three equations all agree in several respects:

First.—The rate of flow increases with temperature.

Second.—The rate of flow increases with the first power of $\frac{h}{L}$, or, if h is constant, varies inversely with L.

Third.—The rate of flow varies with some power of the effective size of the sand grains.

By platting on logarithmic paper, a straight-line relation will be found to exist between Slichter's values of k and the porosity of the material. The same relation holds between porosity and the tabulated values of the transmission coefficient in Water Supply Paper

* U. S. Geological Survey, Water Supply Papers, Nos. 67 and 140; also, 19th Annual Report, U. S. Geol. Survey, Part II, 1899, p. 295.

† *Minutes of Proceedings*, Inst. C. E., Vol. CLXXXI, p. 15; also, Technical Paper No. 97, Govt. of India, 1902.

‡ For description of King's aspirator, see 15th Annual Report, Agricultural Experiment Station, Univ. of Wisconsin, 1898.

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No. 140. It may be shown that Slichter's tabulated results may be expressed in the following form:

$$Q = \alpha\, p^{3.3}\, d^2\, \frac{h}{L}\, A\, T \qquad (4)$$

where p = porosity and T = temperature correction.

The effect of porosity is taken into account in all the equations. In Equation (1), the values of c vary with the uniformity coefficient, which is an indirect and approximate method of stating the effect of porosity. In Equation (3), the effect of porosity is introduced in the coefficient, c_1.

The writer believes that further experiments are required before a formula for underground flow will be developed which will be even approximately as accurate, for instance, as that for the flow of water in pipes and other conduits. Based on present knowledge, such a formula will have the general form:

$$Q = \alpha\, p^n\, d^m \left(\frac{h}{L}\right)^r A\, T \qquad (5)$$

Our present formulas are applicable to sands up to an effective size of 5 mm. Hazen specifically limits the range of his formula to sands of from 0.10 to 3 mm. effective size. The writer has introduced the exponent, r, in Equation (5) as necessary if the formula is to be applicable to all sizes of material. Thus, for large boulders, there is no doubt that r would approach a value of 0.5, as for flow in channels; and, for extremely fine material, such as silt and clay, r may become greater than unity.

Considering the equations for underground flow, we may, for convenience, combine the factors denoting effect of porosity, effective size, and temperature, and write an equation:

$$Q = \frac{K\, h\, A}{L}, \text{ or } p = \frac{K\, h}{L} \text{ for } A = \text{unity}.$$

The coefficient, K, may be taken as a constant for any given case, as for the gravel stratum under the dam described by the author. The head, h, is also fixed by the storage requirements. The only variables then are q and L. The hydraulic gradient equals $\frac{h}{L}$ and may have an infinite number of values, depending on the variable, L. There is, however, a definite relation between K and the character of the underground material.

The writer fails to grasp the author's meaning, when he refers to the hydraulic gradient of various gravels, as though there was only one possible hydraulic gradient for a given gravel. Thus, on page 7, the author states:

"Having determined the hydraulic gradient of the underground material, a trial design was made to find what dimensions would be necessary in a dam constructed wholly of this gravel." Mr. Petterson.

Again, on page 8: "This combined material was then tested in the tank to determine the hydraulic gradient". On page 15, there is a reference in similar vein.

APPLICATION OF FORMULA FOR UNDERGROUND FLOW TO DESIGN OF DAM.

The underground flow may endanger the dam in two ways: (1) The velocity of flow may be sufficient to carry away with it the finer materials in the gravel, and the so-called "piping" action results. If this keeps on indefinitely, it is only a question of time until the underground stratum becomes so porous that all the water in the reservoir will readily escape, or the stratum will settle, and the superstructure will go down with it. (2) The upward water pressure under the base of the dam may be great enough to overcome the weight of the superincumbent earth, and so-called "blow-outs" occur.

(1) *Stability Against Piping.*—Considering the equation, $q = \frac{K h}{L}$, it is obvious, from what has been already stated, that q, or unit discharge, can only be reduced by increasing L. As the velocity is directly proportional to q, it, too, can only be decreased by increasing L, or, what amounts to the same thing, by increasing $\frac{L}{h}$.*

These coefficients are deduced from observations of existing structures, largely in India. They are not the results of experiments, but of compilations of data on structures which have proved to be safe. The writer cannot understand, then, why there is any lack of precedent in designing dams on porous sand foundations. The Laguna Weir is a notable example of a diversion dam founded on extremely fine silt. He agrees with the author as to the value, and even need, of further experiments, if this branch of engineering is to have a scientific basis, but cannot agree as to the value of experiments on models carrying only 10 in. of water.

A particular need for further investigation is the definite determination of the influence of cut-off walls, penetrating only part way through the porous stratum, in reducing the pressure head. According to the "line of creep" theory, this effect depends only on the depth of the barrier and the length of base, L; or, if L_c represents the depth of

*Tables of safe values of $\frac{L}{h}$, for material from fine silt to boulders, may be found in *Minutes of Proceedings*, Inst. C. E., Vol. CXCVII (1914), Paper No. 4004, by W. M. Griffith, or in a standard book like that of Thomas and Watt on "Improvement of Rivers".

Mr. Petterson.

penetration of this cut-off wall, and h_s the reduction of the pressure head, then

$$h_s = \frac{2\,L_s}{\frac{L}{h}}.$$

The writer believes that the value of h_s depends, not only on L_s and $\frac{L}{h}$, but also on the ratio of L_s to the total depth of the porous stratum. If S represents the depth of the porous stratum, and other symbols are as before, then $h_s = \frac{B\,h}{L}\left(\frac{L_s}{S}\right)^n$ is the form of expression which, the writer believes, presents the value of the lost head. B is a coefficient, and n an exponent.

A point of practical importance is the difficulty encountered in making these cut-off walls impervious. Sheet-piling is assumed by some engineers to be practically impervious, but experienced men point out the tendency for deep piles to spread, and mention the difficulty of preventing this tendency, even with interlocking steel piles. As the work cannot be inspected, there is always uncertainty as to the efficiency of sheet-piling.

(2) *Stability Against Upward Pressure Under the Base of Dam.*—The remedy against this method of failure is obvious. If y = the pressure head at any point, w = the weight per cubic foot of water, and m = the weight per cubic foot of the material in the dam; then wy is the intensity of pressure which is resisted by the weight of superincumbent earth. If z = the height of the dam at any point corresponding to y, then, for safety, mz should be greater than wy.

The foregoing discussion assumes that the material in the dam is impervious. If it is porous, the water plane will rise in the dam to heights corresponding to y. It is necessary, then, that z be greater than y. It is also necessary, of course, that the batter of the down-stream face of the dam be sufficiently flat to prevent sloughing of saturated material. The batter of the down-stream face will generally be sufficiently flat to ensure stability against sloughing, if it is designed in accordance with the requirements mentioned under "Stability Against Piping".

The writer believes that a more logical distribution of material is to make a flat down-stream face, with the up-stream face given the usual batter of earth dams, say 1 vertical to 3 horizontal. The material in the up-stream portion should be the most impervious. The reasons for this are well brought out by George L. Dillman,* M. Am. Soc.

* In his discussion on the paper by M. M. O'Shaughnessy, M. Am. Soc. C. E., entitled "Construction of the Morena Rock Fill Dam", *Transactions*, Am. Soc. C. E., Vol. LXXV, p. 52.

C. E. Such great care need not be taken in the construction of the lower part of the dam, though the material should be considerably more resistant to flow than that of the underlying gravel stratum. In other words, the engineer should see to it that the dam is constructed so that greater resistance to flow is offered at all points by the dam than by the underlying gravel, but the most impervious material should be on the up-stream side. Mr. Petterson.

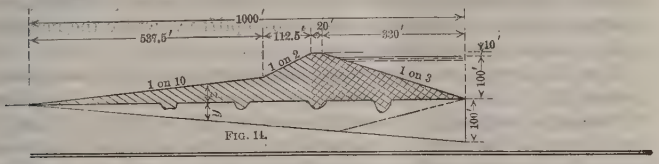

FIG. 14.

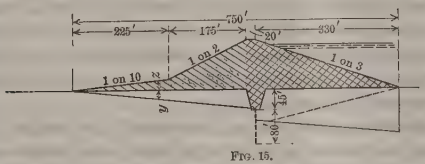

FIG. 15.

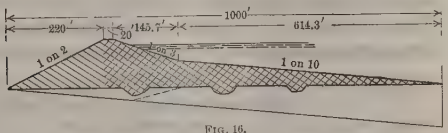

FIG. 16.

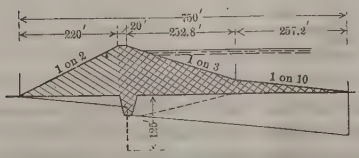

FIG. 17.

The author has reversed the usual method of distribution, in that he makes a steep down-stream face and extends the base up stream. The writer has prepared Figs. 14 to 17 in order to compare the usual design with that of the author. The total width of base in the author's design, as closely as can be determined from his Fig. 9, = 760 ft., and L_s, the depth of vertical barrier, = 125 ft. Then, by the usual

Mr. Petterson.

theory, $L = 760 + 2L_s = 1\,010$ ft. A value of $L = 1\,000$ corresponds to a value of $\frac{L}{h} = 10$, which would be about the value given, in the tables previously referred to, for the gravel described by the author.

Figs. 14 to 17 show four designs with a value of $L = 1\,000$. Fig. 14 shows a section without any vertical barrier constructed in the gravel. The value of m is taken at 100 and that of w at 62.5. The value of z is computed so that $mz = 1.6\ (wy)$ or $z = y$. This gives a factor of safety of 1.6 against failure by uplifting, where the material of the dam is relatively impervious.

Fig. 15 shows the design when an impervious cut-off wall, penetrating 125 ft. into the porous gravel stratum, is used. The effect of this is to increase L, making it $= 2 \times 125$, or 250 ft. Figs. 16 and 17 are for the same general conditions and assumptions, but they have a flat up-stream slope in accordance with the author's idea. The relative quantities of material per unit of length of dam are given in Table 4. The quantities include only those portions of the dams above the natural ground surface. The sections in Figs. 14 and 16 are directly comparable with those for Figs. 15 and 17. Attention may be called to several things in Table 4.

First, as far as total quantities are concerned, the section proposed by the writer has no material advantage over that proposed by the author. The quantity listed as up-stream material is far less, however, for the dams shown by Figs. 14 and 15 than for those shown by Figs. 16 and 17. In view of the foregoing, the dams with the long down-stream slope can be constructed more cheaply.

TABLE 4.—VOLUMES, IN CUBIC YARDS PER LINEAR FOOT, FOR DAMS SHOWN BY FIGS. 14 TO 17, INCLUSIVE.

Up stream.	Down stream.	Totals.
754	876	1 630
754	524	1 278
1 242	448	1 690
841	448	1 289

Second, the dam with the long down-stream slope is likely to be much more efficient in reducing the rate of percolation under the dam. Unless extremely impervious material is secured for the shallow portion of the up-stream section of dams like those of Figs. 16 and 17, it is very probable that there would be percolation downward through the dam, near the heel, adding to the underground flow. In other words, the value of $\frac{L}{h}$ for a dam of this type is less than for a dam of the type shown by Fig. 14, even with the same length of base and the same depth of sheet-piling, or other cut-off wall.

D. C. HENNY,* M. AM. SOC. C. E. (by letter).—The design of a dam in a situation such as the author describes presents an interesting and difficult problem. In his conclusions he mentions several causes of earth dam failures, and states that his paper deals with one cause only, namely, springs or boils, which might produce piping under the base of the dam. Mr. Henny.

The usual percolating velocities are exceedingly low, far below the power of transporting material. The piping or blow-up phenomenon implies a combination of circumstances, relating to the material in place, essentially differing at particular points from the ordinary percolating conditions. In the case of clayey foundation, initial cracks may transmit a large portion of the available pressure to a point close to free exit, and may set up progressive erosion aided by arching. In the case of sand and gravel, strata of unusual openness to flow may be contiguous to layers of very fine sand. In very coarse material, the open spaces between pebbles and cobbles may be so great as to preclude true percolation and permit comparatively free flow and high velocity. The latter case is one which cannot be regarded as applying to foundations for high dams. The other cases are dependent on local deviations from general homogeneity, which for large areas can hardly ever be known definitely. Moreover, the effect of such deviations cannot be ascertained by experiments with selected samples of materials placed in tanks or boxes.

The experiments conducted by the author do not appear to have had for their object the determination of a maximum gradient which would be safe against piping, but rather the most economical form of dam which would produce a maximum reduction of the water gradient, thereby minimizing the piping danger as well as seepage losses.

It appears to the writer that, whatever may be the danger from piping, it must be judged by examination of test pits and experience with existing dams. Usually, with the foundation material which the author describes, and with ordinary slopes of an earth dam, such danger is not great, and if necessary can be counteracted economically by a gravel blanket on the ground below the down-stream toe. Nor need there be any fear of bank sloughing, if gravelly material similar to that in the foundation is used in the down-stream portion of the dam. The real problem seems to be that of insuring against excessive seepage losses such as would render the reservoir useless.

In describing the history of the reservoir, the author states that when the dam was completed to a height of 30 ft., it was subjected to a head of 25 ft., at which time water escaped in considerable quantity from the down-stream toe. Though the quantity of water escaping is not stated, it is evident that, if this is known or can be ascertained ap-

* Portland, Ore.

Mr. Henny.

proximately, a full-size experiment is at hand on which to base some judgment as to the seriousness of the problem.

A portion of this escaping water may have come through the dam proper. It is certain, however, that by far most of it passed through the dam foundation, which is described as being of a very porous nature and of unknown depth, roughly estimated at 240 ft.

If, at that stage of completion, the dam had a full width of base of approximately 500 ft., the water gradient producing this heavy seepage may have been 1 to 20, and a statement of the seepage per linear foot of dam would permit some judgment as to the practical admissibility, from the storage point of view, of such gradient for a full reservoir.

The author's determination of a safe gradient of 1 to 9 by vertical tank test is by no means conclusive. The essential feature, namely, the quantity of seepage with such gradient, is not stated. However, even if it were stated and were satisfactorily low, it is necessarily based on the use of samples of foundation material which, in the nature of the case, cannot represent any known sort of average of the deep masses of gravel, sand, cobbles, and boulders as they lie in place under the dam. Homogeneity cannot exist to any degree in material of greatly varying sizes deposited by successive floods of varying intensity.

Assuming, however, that the samples used in the author's tests are representative of the general foundation, the tests made in a rectangular box with a model of a dam with varying depths of tight cut-off show the quantity of seepage under full head. For the first model tested, this averaged approximately 0.0040 sec-ft., and, for the second model, 0.0025 sec-ft. per lin. ft. of model. The writer understands that these quantities refer to the flow measured in the experiments, on a scale of $\frac{1}{120}$ of full size. If this understanding is correct, then, so far as the experiment goes, the deduction may be made that, for similar material in the foundation, the seepage for a dam built on the basis of the second model would be 120×0.0025, or 0.3 sec-ft. per lin. ft. of dam.

The longitudinal section shows the dam to be 400 ft. in length across the general river bed and 1 600 ft. in length on the adjoining bench. No data are at hand as to rise of rock under this bench. It may be interesting, nevertheless, to inquire as to what the total seepage under the dam would be if the rock were to rise but slightly away from the river. In that case the average gradient and seepage per linear foot for the bench portion of the dam would approximate one-half that for the full height of the dam. On such assumptions, a seepage would result of $\left(400 + \frac{1\,600}{2}\right) \times 0.3 = 360$ sec-ft. This quantity of seepage is clearly inadmissible, and the writer deems it likely that

some of the foregoing assumptions may be known by the author to be erroneous. Mr. Henny.

Independent of the doubt regarding test samples being representative of material in place, there must be serious uncertainty as to the possibility of driving sheet-piling to a depth 80 ft. below the bottom of the cut-off trench, and as to the tightness of such sheet-piling when driven.

It will be noted that the final design of the dam as presented by the author shows an approximate gradient of 1 to 10 for full reservoir. This may be about twice the gradient which prevailed when there was 25 ft. of head against the present dam, at which time heavy seepage losses occurred.

A detailed study of the pressures as registered in the experimental box with the model of the dam reveals some marked inconsistencies, which, if the experimental results are to be made the basis for design, may require explanation. In considering this subject, the writer has confined himself to the use of experiments made under a full head of 10 in. representing 100 ft. on the scale of the experiment, and has selected for this purpose only those numbers, four in the first and three in the second series, for which pressures are recorded at all points.

Individual differences of flow and pressure are rather large, the maximum variations from the average being as follows:

	First series: Experiments 9, 14, 19, and 24.	Second series: Experiments 4, 8, and 12.
Flow	7%	20%
Pressure	20%	13%

In order to eliminate individual variations, whatever may be their cause, pressures and pressure drops are figured on the basis of average values and are listed for comparison in Table 5.

Table 5 shows the following rather surprising results as to pressure head destroyed by percolation: The pressure drop is greater from open water to *A*, from *B* to *C*, and from *D* to *E*, with smaller than with larger flow; the pressure drop is greater from *A* to *B* without sheet-piling, and with small flow than with sheet-piling and with large flow; the pressure drop is greater from *C* to *D*, in proportion to the flow, with shallow than with deep sheet-piling.

The drop from open water to the point, *A*, up stream from the points of cut-off, is 60% of the total head, in spite of the short distance of travel; so that the upper portion of the water gradient is steepest. The great loss of head at entrance appears to be inherent in experiments of this kind. It may well be doubted, however, whether such losses occur in the case of actual dams, where the area of entrance is very extensive, unless it is induced by silt deposits. Should no such

Mr. Henny.

TABLE 5.—Pressures and Pressure Drops.

	Average of Experiments.			
	1st Series: Nos. 9, 14, 19, and 24.		2d Series: Nos. 4, 8, and 12.	
Measured flow	0.0040 sec-ft.		0.0025 sec-ft.	
	Pressure.	Pressure drop.	Pressure.	Pressure drop.
Water toe	100 ft.		100 ft.	
		58.4 ft.		60.5 ft.
A	41.6		39.5	
Cut-off	50 ft. in 240 ft.	4.0	25 ft. in 240 ft.	11.8
B	37.6		27.7	
		2.8		5.7
C	34.8		22.0	
Cut-off	85 ft. in 240 ft.	21.9	125 ft. in 240 ft.	17.7
D	12.9		4.3	
		2.8		3.7
E	10.1		0.6	
		2.2		0.6
F	7.9		0	
		92.1		100.0

loss with the actual dam be experienced, and should the pressure at *A* be 80 or 90% of the total head instead of 40%, the actual seepage losses may be double those deduced from the experiments.

In regard to the feature of the design consisting of a tight blanket up stream intended to lengthen the path of the water, the reasoning of the author is believed to be sound. The same method was advocated by the writer and was adopted in the case of the Grand River Diversion Dam built by the Reclamation Service near Grand Junction, Colo. The object in this case was the reduction both of uplift and of seepage. Fig. 18 shows a cross-section of this dam.

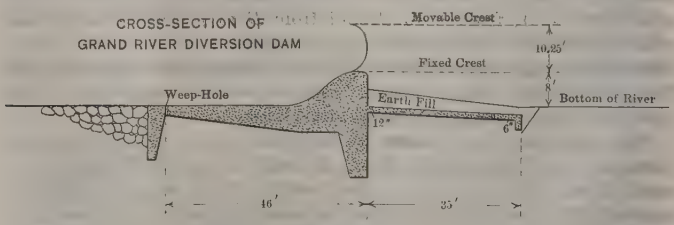

Fig. 18.

It may be stated that in this case measurements of uplift pressures were made through pipes placed in piers in the dam and ending in

pockets of screened gravel under the foundation. The results indicate complete absence of entrance losses. They also show drop of pressure to be closely proportionate to distance along line of creep. It should be stated, however, that owing to delay in placing movable gates, the heads at the time of the two measurements were in each case only between 4 and 5 ft., and that measurements under a full head of 18 ft. may give different results. Mr. Henny.

Experiments of the character made by the author must always be of intense interest to hydraulic engineers, and deserve full recognition.

JOSEPH JACOBS,* M. AM. SOC. C. E. (by letter).—This paper is of interest because it deals with an important engineering subject, concerning which there is all too little reliable information on which definite theory and judgment may be predicated. The author is entitled to credit for assembling the results of these tests for the use of the Profession, and for his frankness in presenting his interpretation and application of the facts disclosed. Mr. Jacobs.

The writer believes that the author failed to reach certain important conclusions, apparently fully warranted by the tests, and that in some respects his deductions as to the theory of underground water movements are fallacious. Particularly does he believe that the tests do not justify the conclusion, reached by the author, that the "line of creep" theory has been disproved.

In describing the initial seepage test in the 30-in. cylindrical tank (page 7), the author states:

"A valve at the bottom held back the water until the pore spaces were completely filled; then the valve was opened, and after a few minutes the relative elevation of the water in the two tubes became constant. The difference in elevation determined the loss of head, and from this the hydraulic gradient was computed."

* * * * * * *

"With 6 in. of water on the soil surface, and 8 in. of soil above the upper tube, the loss of head was 1 ft. in 9."

* * * * * * *

"Having determined the hydraulic gradient of the underground material, a trial design was made to find what dimensions would be necessary in a dam constructed wholly of this gravel."

These quotations, it seems to the writer, disclose a fundamental error in the author's application of his test. It will be noted that there is no mention of the quantity of seepage flow, and, as seepage flow and hydraulic gradient are inter-dependently related, the mention of one means nothing unless there is also included a definition of the other. The hydraulic gradient of 1:9, as found by the author, would appear to be a purely fortuitous result, for, with a variation in the size

* Seattle, Wash.

Mr. Jacobs.

of valve opening (if it is less than the capacity of the gravel), and with variation in the depth of water above the soil surface, the ratio of loss of head to length of travel may be made to vary indefinitely. The maximum permissible limit of this ratio is that at which the seepage loss becomes prohibitive as to quantity, or at which the induced velocities are such that erosion or piping is threatened as a forerunner of the ultimate failure of the dam. It may be that the author decided that these requirements had been met in his hydraulic gradient of 1:9, but there is nothing in the text indicating this special consideration, nor do the seepage tests on the model dams seem to warrant such a decision.

It will be noted from the Second Series of Tests, which relate to the model from which the final dam was designed, that under a 10-in. head there was a mean seepage loss of approximately 0.0025 cu. ft. per sec. per lin. ft. of model dam. Bearing in mind that the model dam was built on a scale of 1:120 as compared with the final dam, and assuming, for the moment, that the hydraulic gradients and therefore the seepage velocities would be identical in the two, it follows that the seepage loss for the actual dam, under a condition of full reservoir, would be 360 cu. ft. per sec., made up as follows:

400 lin. ft. of dam under 100 ft. head, yielding.....120 cu. ft. per sec.
1 600 " " " " " 100 to 0 ft. head "240 " " " "

Total rate of seepage flow at full reservoir.....360 cu. ft. per sec.

This 360 cu. ft. per sec. would be further increased by reason of the hydraulic gradient in the actual dam being greater than that in the model. In comparing ratios of head to length of water travel, it is the friction head and not the total head that must be considered. In the model dam the 10 in. total head is greatly depleted by entry and exit heads, thus leaving a greatly reduced head available for friction losses, whereas, in the actual dam of 100 ft. head, the same quantitative depletion for entry and exit heads has relatively slight effect on the remaining friction head.

So great a loss as 360 cu. ft. per sec. from any reservoir would, of course, be prohibitive, even in the absence of any danger of undercutting the dam, and it must be assumed that the figures given in the paper are in error or that the author failed to apply the apparently clear inference to be drawn from his tests, which latter seems to be untenable because the tests appear primarily to have been suggested by the seepage difficulties encountered in the original dam. It would be interesting to have now a record of the actual behavior of the dam, as to seepage losses, if there has been opportunity for test since construction.

Mr. Jacobs.

Concerning entry head and exit head there is indication that the total head is affected by these elements, and it is this fact that renders difficult of interpretation the results obtained from experiments with small model dams, for in such dams the combined entry and exit heads may indeed form a relatively large part of the total head, though in dealing with full-sized dams they would form a relatively small or negligible part. To determine entry and exit heads from any set of tests, such as those by the author or by Mr. Colman,* we must first ascertain the friction head, and this in turn depends on the assumption as to line and resultant length of water travel along any film or differential element of flow. A critical examination of these tests shows a rate of loss of head due to friction that is not uniform per distance traveled, as measured between observed tubes. This disparity is due no doubt to variation in porosity and other mechanical elements of the pervious medium, and probably to some extent to observational errors, for it has been well established that, for an unchanging medium of this character, the rate of friction head consumption will be uniform. On the scheme of underground water movement, as hereinafter explained, the writer finds values of entry and exit heads in the Hays and Colman tests varying as follows:

In the Hays tests, with a total head of 10 in., entry heads varied from — 0.90 to + 5.98 in., the mean being + 5.5 in.; and exit heads from — 2.26 to + 0.45 in., the mean being — 0.9 in.

In the Colman tests, with an average total head of 54 in., entry heads varied from — 1.08 to + 21.36 in., the mean being + 7.0 in.; and exit heads from — 7.32 to + 13.08 in., the mean being + 3.4 in.

Because velocity is a function of all the elements which characterize the medium through which the water moves, it was thought that some relationship might be established between entry head and entry velocity, and exit head and exit velocity. The graphic method was used, but there was such lack of consistency in the results deduced from the tests that no dependable curves could be platted. Nor was this lack of consistency due to the particular scheme of water movement assumed, for other assumptions as to line and length of water travel showed similar inconsistencies. In the absence of more extensive data than we now have, we may only say at this time that there is entry head and exit head of uncertain amounts which must be reckoned with in experiments on small models, but which are negligibly small in dealing with full-sized dams.

All the formulas for the flow of water through soils are of the general form, $Q = c \frac{h}{l} a$, in which

* *Transactions*, Am. Soc. C. E., Vol. LXXX, p. 421.

Mr. Jacobs.

Q = discharge;

a = area of cross-section;

$\frac{h}{l}$ = ratio of loss of head due to friction to length of water travel;

and c = a variable coefficient, dependent on water temperature, porosity, and mechanical analysis of the soil.

Water will follow the line of least resistance, and therefore that form of section and route of water movement will obtain which, for a given head, will afford a maximum discharge, or which, for a given discharge, will consume the minimum head. These conditions, as will readily be seen from the foregoing equation, in which c for any specific case is a constant, will be satisfied when $\frac{l}{a}$ is a minimum; and, in studying seepage and pressure problems, with respect to dams on porous foundations, we must first ascertain the form of section and route of water travel which renders $\frac{l}{a}$ a minimum.

It cannot be assumed that the water will be drawn from an indefinite distance up stream, nor released throughout an indefinite distance down stream, from the dam, for, if that were the case, Q would approach its minimum rather than its maximum, and $\frac{l}{a}$ would approach its maximum rather than its minimum value.

There may be, and doubtless is, some seepage draft from points far above, and seepage delivery to points far below, the dam, but the quantities would be negligible in comparison with the main body of seepage water, which, in the writer's opinion, based on mathematical analysis, would necessarily be drawn from, and delivered to, points in the immediate vicinity of the dam. The lengths of river-bed over which water draft and water release are operative, with respect to any dam, are important, because these lengths fix the hydraulic gradients on which pressure and seepage discharge depend.

The actual form of approach and exit channel for seepage, from free water above to free water below the dam, will depend on the base width of the dam, the depth of the pervious medium, the position of the cut-off wall, and the ratio of the depth of the cut-off wall to the depth of the pervious medium. If the approach is from free water above the dam to a full vertical section beneath the heel of the dam, the writer finds that, whatever its form, the minimum value of $\frac{l}{a}$ occurs when the distance up stream from the dam equals the depth of the pervious medium beneath the dam; that, of the various curves

Mr. Jacobs.

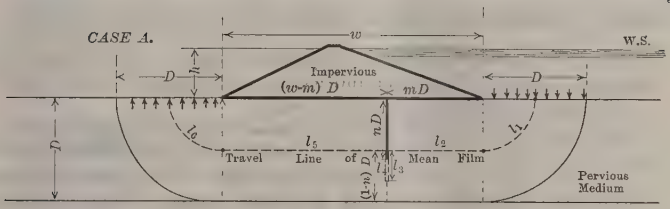

FIG. 19

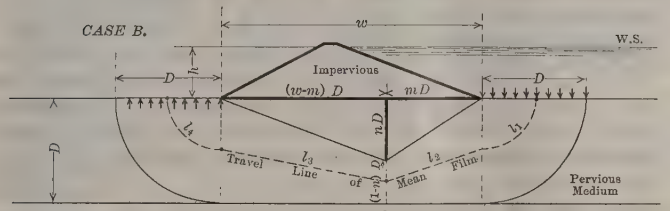

FIG. 20

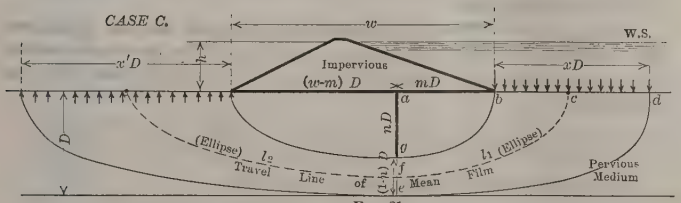

FIG. 21

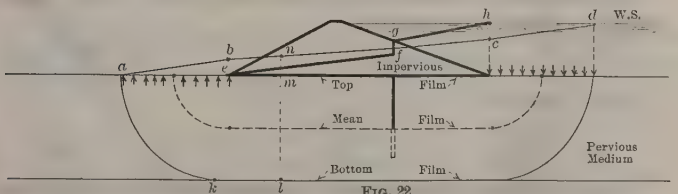

FIG. 22

Mr. Jacobs.

which might define the form of approach, the ellipse affords the minimum $\frac{l}{a}$; and, finally, that the circle, as a special form of ellipse, affords the absolute minimum. This, then, means a circle having its center at the heel of the dam and having a radius equal to the depth of the pervious medium beneath the dam.

In Figs. 19, 20, and 21 are shown three forms of seepage channel, any one of which, assuming that the dam itself is impervious, might be the actual for a particular case, depending on the position of the cut-off wall and other conditions, as already stated. These will now be examined: Referring to Figs. 19 to 21, consider a section of unit thickness normal to the paper. Dividing the seepage channel into longitudinal sections, as obviously demanded by their change in cross-sectional form, the mean lengths of these sections would be l_1, l_2, etc., as shown, and, for a unit thickness, their mean areas would be some function of D, the depth of the pervious medium beneath the dam. The total seepage flow would be the aggregate of the approximately parallel films of flow, each film having its individual length, individual discharge, and individual hydraulic gradient. The mean film would control as to the total seepage loss, and the hydraulic grade line of the top film would control as to the pressure on the base of the dam, except that, at and near the lower toe of the dam, in some cases, the hydraulic grade line of the bottom film might control, as will be explained later.

Case A.—In this case the seepage flow occupies the full prism section between the base of the dam and the base of the pervious stratum. The mean lengths of its several sections, as is obvious from Fig. 19, would be:

$$l_1 = l_6 = 0.7854\ D,\ l_2 = m\ D,\ l_3 = l_4 = \frac{n\ D}{2},\ l_5 = (w - m)\ D.$$

The mean areas of these sections would be:

$$a_1 = a_2 = a_5 = a_6 = D;\ a_3 = a_4 = \frac{D + (1 - n)\ D}{2} = \frac{D}{2}(2 - n).$$

From the original equation for seepage flow, $Q = c\,\frac{h}{l}\,a$, we obtain, $h = \frac{Q}{c} \times \frac{l}{a}$; or, for any section, n, $h_n = \frac{Q}{c} \times \frac{l_n}{a_n}$. The total head, therefore, would be, $h = h_1 + h_2 + \ldots . h_6$

$$= \frac{Q}{c}\left(\frac{l_1}{a_1} + \frac{l_2}{a_2} + \ldots . \frac{l_6}{a_6}\right) = \frac{Q}{c}\left(1.5708 + w + \frac{2\ n}{2 - n}\right).$$

Case B.—In this case (Fig. 20) the seepage flow occupies the wedge-shaped sections beneath the dam, and, in the space between the base

of the dam and the top of the seepage channel, there would be dead water, or approximately dead water. For this case we would have: Mr. Jacobs.

$$l_1 = l_4 = 0.7854\ D,\ l_2 = \frac{D}{2}\sqrt{4\ m^2 + n^2},\ l_3 = \frac{D}{2}\sqrt{4\ (w - m)^2 + n^2}.$$

$$a_1 = a_4 = D,\ a_2 = a_3 = \frac{D}{2}\ (2 - n).$$

$$h = (h_1 + h_2 + \ldots\ h_4) = \frac{Q}{c}\left(\frac{l_1}{a_1} + \frac{l_2}{a_2} + \ldots\ \frac{l_4}{a_4}\right)$$

$$= \frac{Q}{c}\left(1.5708 + \frac{\sqrt{4\ m^2 + n^2} + \sqrt{4\ (w - m)^2 + n^2}}{2 - n}\right).$$

Case C.—In this case (Fig. 21) the form of approach and exit is defined by ellipses, as shown, and, as in the previous case, the space between the base of the dam and the top of the seepage channel would be approximately dead water. The distance up stream from which seepage water would be drawn is determined as follows:

The mean film, $l_1 = fc$, is the quarter circumference of an ellipse the major and minor semi-axes of which are $a\ c = \frac{D}{2}\ (2\ m + x)$ and $a\ f = \frac{D}{2}\ (1 + n)$, respectively, and its length, according to the usually accepted approximate formula, would be

$$l_1 = \frac{\pi}{4}\frac{D}{2}\sqrt{(2\ m + x)^2 + (1 + n)^2}.$$

The cross-sectional area of the seepage channel, for unit thickness, would vary from $bd = xD$ to $ge = (1 - n)D$, and the mean area would be the area, *bdeg* $\div\ l_1$, the area, *bdeg*, being equal to the difference between the areas of the elliptical quadrants, *aed* and *agb*. The area, *bdeg* $= A = \frac{\pi}{4}\frac{D^2}{}\ (m + x - m\ n)$, and $a_1 = \frac{A}{l_1}$. Therefore,

$$h_1 = \frac{Q}{c} \times \frac{l_1}{a_1} = \frac{Q}{c} \times \frac{l_1^2}{A} = \frac{Q}{c} \times \frac{\pi}{8}\left(\frac{(2\ m + x)^2 + (1 + n)^2}{m + x - m\ n}\right) \ldots (1)$$

To find the minimum value of $\frac{l_1}{a_1}$, differentiate and place the first differential coefficient equal to zero, thus,

$$\frac{\delta\ h_1}{\delta\ x} = \frac{Q}{c} \times \frac{\pi}{8}$$

$$\left(\frac{2\ (m + x - m\ n)\ (2\ m + x) - ((2\ m + x)^2 + (1 + n)^2)}{(m + x - m\ n)^2}\right) = 0.$$

Mr. Jacobs.

Clear of fractions, transpose, and reduce, and we have,

$$x^2 + 2m\,(1 - n)x = (1 + n)^2 + 4m^2n.$$

Complete the square, and we have,

$$x^2 + 2m\,(1 - n)x + m^2\,(1 - n)^2 = (1 + n)^2\,(1 + m^2).$$

Therefore,

$$x + m\,(1 - n) = \pm\,(1 + n)\,\sqrt{1 + m^2},$$

whence

$$x = (1 + n)\,\sqrt{1 + m^2} - m\,(1 - n),$$

and this critical value of x proves to be the minimum rather than the maximum value of $\frac{l_1}{a_1}$. We would also have,

$$x' = (1 + n)\,\sqrt{1 + (w - m)^2} - (w - m)\,(1 - n).$$

Applying these values of x and x' to Equation (1), and reducing, we would have

$$h = h_1 + h_2 = \frac{Q}{c}\left(\frac{l_1}{a_1} + \frac{l_2}{a_2}\right)$$

$$= \frac{Q}{c} \times \frac{\pi}{4}\,(1 + n)\left[w + \sqrt{1 + m^2} + \sqrt{1 + (w - m)^2}\right].$$

Which one of the foregoing three cases would obtain for any actual dam would depend on the values of m, n, and w, but it would always be that one case, or that combination of the several cases, which would render $\frac{l}{a}$ a minimum. To determine this for any particular set of conditions is a simple computation, but a tabulation for varying values of m, n, and w, shows that, in most instances, Case A will obtain, and that Case C obtains only when m, n, and w are small. As between Cases A and B, when m is less than $0.66\frac{2}{3}$, the wedge affords the minimum $\frac{l}{a}$; when m is greater than 1.00, the prism affords the minimum $\frac{l}{a}$; and between these values of m, it may be either wedge or prism, depending on the value of n.

Now consider the question of the hydraulic upward pressure against the base of the dam: If the total head is known (as it would be in the cases considered), then its apportionment among the several sections of the seepage channel results from the general equation,

$$h = (h_1 + h_2 + \ldots\ldots h_n) = \frac{Q}{c}\left(\frac{l_1}{a_1} + \frac{l_2}{a_2} + \ldots\ldots \frac{l_n}{a_n}\right),$$

and would be as

$$\frac{l_1}{a_1} : \frac{l_2}{a_2} : \ldots\ldots \frac{l_n}{a_n}.$$

Mr. Jacobs.

This apportionment, it will be noted, is independent of Q and c, for Q is necessarily the same for all sections, and c, too, is a constant, if the pervious medium be regarded as homogeneous throughout. If the medium is not homogeneous, and c is variable from section to section, then that factor would need to be retained in the computation, but the apportionment of head would still be fully determinable, if c were known for each section. This lack of perfect homogeneity tends to confuse the results in any study of the laws of underground water movement, and it is no doubt partly responsible for the variations and apparent disparities noted in the Hays' experiments.

In the top film of a seepage channel, the form of the channel having been determined, the lengths, l_1, l_2, etc., become known, and, as for the areas, a_1, a_2, etc., it must be assumed that, in this differential element of flow, the differential areas of the several sections bear the same relationship to each other as do the like whole areas of the whole seepage channel. The relative values, $\frac{l_1}{a_1}, \frac{l_2}{a_2}$, etc., thus become known, and the apportionment of the total head among the several sections of the top film, according to the method outlined in the previous paragraph, readily follows. From this determination of the heads, h_1, h_2, etc., the hydraulic grade line for the top film (and in similar manner for the bottom film) can be constructed, whence the computation for the hydraulic upward pressure proceeds by the proper evaluation of the area included between base of the dam and the higher of these grade lines; that for the bottom film, however, is subject to slight modification.

Referring to Fig. 22, it will be noted that *abcd* is the hydraulic grade line for the bottom film, and *efgh* for the top film, and that, for the section of the dam below the cut-off, the former appears to control as to the hydraulic upward pressure against the base of the dam. This is not necessarily so, however, nor do these hydraulic grade lines indicate the exact plane to which water would rise if the down-stream portion of dam were pervious, for there would be loss of head and pressure between the film considered and the final water plane, wherever it might be. The ordinate, m n, for instance, measures the head or hydraulic pressure of the bottom film at the point, l, whereas the actual pressure at m would be less than m n because of the loss of pressure between l and m, and because otherwise the water would tend to find escape in that direction rather than along the course, *lka*. The actual pressure line for the base of the dam due to the bottom film is probably little, if any, above that due to the top film, but the writer does not, at this time, know just how it can be located with assurance of entire accuracy.

Mr. Jacobs.

It will be noted that the validity of the "line of creep" theory is herein recognized, but that it does not have universal application. It obtains with respect to Case *A* when considering hydraulic pressures against the base of the dam, but does not apply to Cases *B* and *C*. In his examination of the author's tests (Case *A* applies), the writer finds nothing to upset the general theory developed in the foregoing, but, on the contrary, finds some confirmation in the fact that the average maximum departure of the observed from the writer's computed hydraulic grade line was only $\frac{3}{4}$ in. When it is considered that the author's tests show a computed variation in entry head of from — 0.90 to + 5.98 in., and in exit head of from — 2.26 to + 0.45 in., a departure of $\frac{3}{4}$ in. from a computed grade line should occasion no surprise. The uncertain effect of capillarity, undefined entry and exit resistances, variations in the mechanical analyses of the soil, and slight observational errors, would readily account for so inconsiderable a discrepancy.

On the whole, it may be questioned whether results from tests on such small-size models are of great value. The disturbing and relatively important effect of exit and entry heads, the possibility of inadequate box length to provide free approach and free exit of the seepage water, and other inherent limitations, certainly make the correct interpretation of the results extremely difficult. In the writer's opinion, the only acceptable confirmation of any theory as to the movement of seepage waters beneath dams, and the effect of cut-off walls thereon, must come from observation of full-sized structures actually built.

In closing, the writer would add that the following necessary conclusions are deduced from the general theory herein developed:

1.—That the form of computation for determining discharge and hydraulic grade line are similar to and practically as simple as those for pipe lines. With a reasonably accurate selection of the coefficient, c, in the first general formula, a reasonably accurate discharge, Q, can be predicted for any head of water above the dam, and a reasonably accurate hydraulic grade line can be platted for computation of pressures.

2.—That the cut-off wall must extend practically to an impervious stratum in order to reduce seepage loss appreciably. This fact is important in cases where seepage, by reason of its quantity, or its possible erosive action, must be considered.

3.—That, from considerations of both seepage loss and reduced hydraulic pressure on the base of the dam, the cut-off wall should be as near the heel of the dam as the local physical conditions will permit.

JAMES B. HAYS,* JUN. AM. SOC. C. E. (by letter).—The deep interest taken in the paper is ample evidence that many engineers are always on the alert for any bit of information pertaining to this subject. All members of the profession will welcome any information based on fact, either from field experience or models; although the former is greatly to be preferred, the latter will furnish subject matter for deep study in almost any case, aiding the designer to base his judgment on something more than guesswork. The writer's object, in making this model and testing it, was to apply the results, not directly, but toward forming some definite theory regarding the flow of water under dams. Mr. Hays.

On one point all the critics of the paper seem to agree in objecting to the results, that point being in regard to the size of the model, etc. There is only one factor that might vary with a larger model as compared with the small one, and that is in regard to the loss of head due to the water entering the soil.

The writer would earnestly suggest to all interested to try the method used in these experiments for obtaining the correct proportions of soil for the impervious section, whenever the occasion or chance should arrive. An extremely dense mixture will be obtained, and this, when properly placed, will hardly leave room for the hygroscopic moisture. When once saturated, the mixture will remain so for a long period, even though there is no water above the dam to force the water into it. This was the writer's experience. After soaking the model completely, the results did not seem to change, whether or not the model was left with a full head of water for 24 hours.

The writer has due respect for the "line of creep" theory, and realizes that many excellent structures have been built on that basis; but it certainly did not hold good for the upper row of sheet-piling, when the design is considered part by part; and, at the same time, the lower row of piling produced excessive results, which certainly were not caused by the upper row of piling being in place, other than that the total results were lower than if the lower row of piling had been the only one used. In other words, although the total results approached the "line of creep" theory, why did not either row of piling show a loss of head based on the same assumption? The only answer to this question, in the writer's opinion, is in the partly closed valve theory.

Imagine a pipe line with two valves; close the lower one nearly half way, and then close the upper one only one-eighth of the whole distance. The discharge is lessened, the velocity through the main portions of the pipe is lessened as a result, but the lower valve (corresponding to the lower sheet-piling) creates no small loss of head (assuming that the water is passing through the pipe and not standing still), which is represented by a high velocity through the valve. The upper valve does not have its full effect, does not produce the loss of head that it would were

* Boise, Idaho.

Mr. Hays. the lower valve not closed. The reason is that the water velocity is lowered by closing the lower valve for the entire pipe, consequently the quantity is less, and hence a lower velocity or loss of head is required at the upper valve to pass the smaller quantity of water, when compared with the original discharge. It is quite evident that the upper valve does not have the same effect on the lower one, although it does assist in lowering the total discharge. If the lower valve were closed still more, by an amount equal to the closing of the upper valve, and the upper valve were opened, the total loss of head would be greater than for the two valves partly closed, as just described.*

Extend the piling cut-off deep enough to leave only a very small opening between the bottom of the pile and bed-rock, and we have, horizontal 745 ft., vertical $2 \times 239 = 478$ ft.; then the total creep is 1 223 ft., and the gradient is 1:12.23. The loss of head due to the piling would be about 39 ft., whereas the actual loss of head could not fail to be far greater than the above figure. Extend the piling still deeper, leaving an infinitesimal opening under the cut-off for the water, and the results will depart still farther from the "line of creep" theory.

It was for this reason that the writer discarded this theory, when it failed to hold out, and accepted the "partly closed valve" theory, which explained thoroughly the results obtained with the model.

There must be a limit to the "line of creep" theory somewhere, but it is not mentioned in any of the standard works discussing the subject. Weisbach's experiments show that a gate-valve closed seven-eighths produces a loss of head nearly 47 times as great as when closed only half way, the "line of creep" theory, under the same conditions, shows that the loss of head due to closing down seven-eighths is only one and one-half times the loss produced by closing half way.

That the leaky condition of the upper row of sheet-piling should be plainly visible from a great distance, and not to the experimenters, is strange. Attention is called to the notes below Tables 2 and 3.

The low heads reported for the points, H to K, were taken simultaneously with the readings for the other tubes, and the drain valve was closed, during all experiments and between days, and the leakage from the tank was nil.

All gravel larger than 1½ in. in diameter was removed from the material placed in the bottom of the tank, and 1 in. was the limit allowed for material in the section of the model. This was done in order to eliminate leakage surfaces for the water to follow, as they would be all out of proportion to the size of the model.

The quantity of water entering the vertical circular tank was the same as that leaving the drain at the time the readings were made. Time was allowed for the water to adjust itself to the conditions.

* "Treatise on Hydraulics", by Merriman, Art. 88; also *Transactions*, Am. Soc. C. E., Vol. XXVI (1892), p. 449.

Mr. Hays.

Lowering the head on top of the soil in the vertical tank caused the water to lower in both tubes by the same distance. It was determined definitely that the water was not held up in the tank sufficiently to affect the readings of the lower tube at any time.

Water did not flow through the drain valve during the experiments on the model, as Mr. La Rue seems to have understood; this would have defeated the experiments entirely. Instead, the water-table was forced up to within a short distance of the ground surface below the dam to represent the actual conditions, as nearly as possible. Practically all the material comprising the foundation of the dam at the actual site is saturated. There is a definite underground stream flowing down the valley.

Where the upward pressure against the base of the dam was mentioned by the writer, it was meant to be inferred that it was the pressure on an imaginary plane, just as one would speak of the hydrostatic pressure at a certain point in a body of water.

Evidently Mr. Petterson disregards the writer's idea of making the down-stream section of the dam more stable by having a very small upward pressure beneath that portion, whereas, in designs such as Fig. 14, a very high percentage of the material in the down-stream section is thoroughly saturated, because the hydraulic gradient is close to the surface. Assuming in both cases that a cut-off wall through the main portion of the structure will cause all percolation water to come under the dam, the hydraulic gradient is in both cases projected at a slope of 1:10, beginning at the toe and extending up stream. In Fig. 15 the lower 225 ft. of the down-stream section just covers the hydraulic grade line. Mr. Petterson has taken his hydraulic grade line below the base of the dam apparently to indicate the upward pressure on the base, or the imaginary plane which we call the base. The water will rise in the body of the dam to an extent governed by the upward pressure at any point. Where y and z or y' and z' are respectively equal or nearly equal, it is readily seen that the water will rise in the body of the dam to the line as shown, z or z' above the base. Thus, the hydraulic grade line for the lower portion of the down-stream section of the section shown in Fig. 15, will be identical with the surface of the material constituting the dam. In an earth dam this would certainly be disastrous. Continue the gradient at the rate of 1 on 10 for the next 175 ft. under the portion with the 1 on 2 slope, and it will be observed that a large portion of this down-stream section will undoubtedly become saturated. Of the down-stream section of Fig. 15, it is computed that 453 cu. yd. per lin. ft. of material will be dry, that is, above the hydraulic gradient (1:10). Fig. 17, which corresponds to the writer's second section, shows that 717 cu. yd. per lin. ft. of material in the same section will be dry. Mr. Petterson's argu-

Mr. Hays. ments on this point convince the writer, more than ever, of the advisability of having as much material as possible in the down-stream section dry, in order to secure stability. Figs. 14 and 16 can be compared in the same manner, showing even greater advantages in favor of the latter, as very little of the material in Fig. 14 will lie above the hydraulic gradient. Material below the hydraulic gradient will lose about 50% of its weight, which might be exerted toward increasing

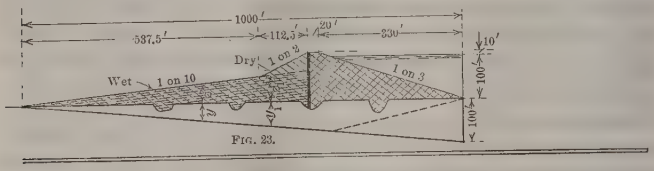

FIG. 23.

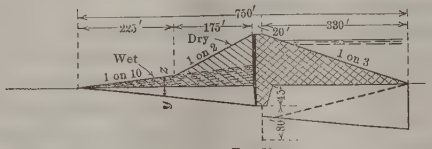

FIG. 24.

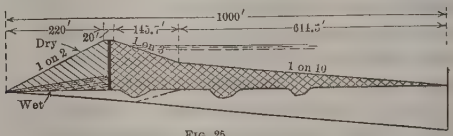

FIG. 25.

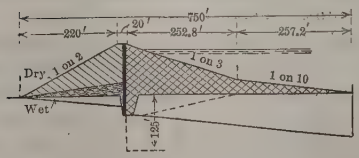

FIG. 26.

stability, due to buoyancy. Using Figs. 14 to 17, the writer has prepared Figs. 23 to 26, on which the hydraulic gradients are drawn through the down-stream section, in order to show the dry and saturated sections in each case.

Regarding the possible underflow to be expected under the finished structure, it would seem that several engineers have jumped to conclusions in this matter. Assume that the water flows through the

foundation material of the dam in the same manner as through a large bundle of very small pipes. These pipes will have such a small diameter that capillary action will have a great effect proportionately, and the velocity of the water will be so low that the discharge will vary directly as the first power, rather than the square—this being an elementary rule in hydraulics. The length of flow and the increased head in the proposed final structure will be proportionate, hence there will be the same loss of head per unit of distance along the base of the dam; but multiplying the area through which the discharge takes place will produce different results from those obtained by taking the required number of bundles of pipes. Two pipes having the same total area as a larger pipe will not have the same capacity as the larger pipe—assuming, of course, that the loss of head per unit distance is the same in each case. The velocity is higher in the larger pipe, with the same loss of head per foot, than in the two smaller pipes. Mr. Hays.

Many of the criticisms of the tests are well taken, but the writer hoped for more discussion containing real meat, rather than theory. It will undoubtedly be a long time before anything definite can be decided which will furnish a sound basis for sheet-pile cut-off walls, under similar conditions, other than the information now available. As mentioned previously, the "line of creep" theory is not yet to be discarded, but it has not yet been practically proved, except as regards the fact that dams, well built, and following that theory, have stood the test. Actual records of the loss of head due to piling seem to be lacking, and this is what the writer wanted to know when the tests on the model were attempted. He is firmly convinced that tests on a large model of the same structure would show similar results, aside from the entry head.

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Paper No. 1380

UNDERPINNING TRINITY VESTRY BUILDING FOR SUBWAY CONSTRUCTION*

By H. de B. Parsons, M. Am. Soc. C. E.

With Discussion by Messrs. James C. Meem, Elias Cahn, T. Kennard Thomson, James F. Fouhy, Charles Rufus Harte, Joseph A. A. Connelly, J. S. Branne, A. W. Buel, and H. de B. Parsons.

Synopsis.

This paper describes the underpinning of the Vestry and School Building of the Corporation of Trinity Church, which was made necessary by subway construction.

The building is on Church Street, between Fulton and Vesey Streets, Borough of Manhattan, New York. It is a brick structure with thick walls, having four floors, basement, and part attic, and was constructed about 30 years ago.

The Church Street subway curves at Fulton Street and passes beneath the building and beneath the Churchyard of St. Paul's Chapel to Vesey Street. The subway structure consists of two circular tunnels, each 20 ft. in diameter. The tops of these tunnels are just beneath the level of the footing stones of the building. As the tunnels run diagonally beneath the building, it was necessary to remove its original foundations entirely, to construct new foundation piers, and to transfer the load of the building to these new piers by means of cross-girders.

* Presented at the meeting of October 4th, 1916.

The paper describes how this work was done, and how the new foundation piers were sunk so as to cause the least disturbance to graves and vaults.

The brick walls of the building were first strengthened by fastening to them heavy lattice girders on each side, and, on the Fulton Street face fastening two 60-in. plate girders in a similar manner.

Excavations for the new underpinning piers were made at selected intervals, so that the settlement of the building could be kept under control.

The paper describes the use of concrete underpinning piles, and gives the results of a number of tests made to determine their bearing capacities. These tests showed that the piles had a tendency to rebound when pressure was relieved, and the paper describes how this element of weakness is overcome when such piles are used for underpinning.

The method of tunneling through the sand beneath the Churchyard and building is described.

Surveys were made before the work was undertaken and after it was finished, and these showed that the subway construction work caused the building to settle approximately 2 in., which was fairly uniform on account of the manner in which the underpinning piers were sunk.

The contract price for 1 000 ft. of this subway construction work was $982 740. The work commenced in June, 1913. Both tunnels were completed in June, 1914. The basement floor of the building was restored to the owners by October, 1914. The final restoration of the whole property was practically finished in April, 1916.

As the underpinning of the Vestry and School Building of the Corporation of Trinity Church was an unique example of protection work, made necessary by subway construction in New York City, the writer presents the following description which he hopes will be found of interest by many.

The chief features of this underpinning were the entire removal of the original foundations, the construction of new foundation piers to an elevation below the sub-grade of the tunnels, and the transference of the load of the building to these new piers, without interfering with its occupancy or use.

The building is on the west side of St. Paul's Churchyard, and faces on Church Street between Fulton and Vesey Streets. Fig. 1 is a general view of the building, which shows clearly its chief characteristics and the proximity of the Churchyard, with its graves and vaults.

The building is of brick, having four floors, basement, and a part attic. It has a gabled roof, with side gables, or dormers, all having slopes of 45 degrees. The walls are 2 ft. thick above the water-table, and 2 ft. 9 in. below that level. The floor-beams are supported on the outside walls. In addition to these outside walls, about 100 ft. of interior walls also carry weight. The floors are surfaced with wooden or mosaic flooring, according to the use of the various rooms. The building is of heavy construction, and was completed about 30 years ago.

The subway, known as Route 5, was designed to run north on Church Street as far as Fulton Street, then to curve eastward beneath the Vestry Building and the Churchyard to Vesey Street, and then to curve again to the northward into Broadway. A special agreement was entered into between the Vestry of Trinity Church and the Public Service Commission, representing the City of New York, whereby the Vestry gave the City, under certain restrictions, a right of way for the purpose of constructing tunnels for a rapid transit railway beneath the Church property. The right of way was deeded to the City, as the Vestry realized that this subway work was part of a great municipal improvement.

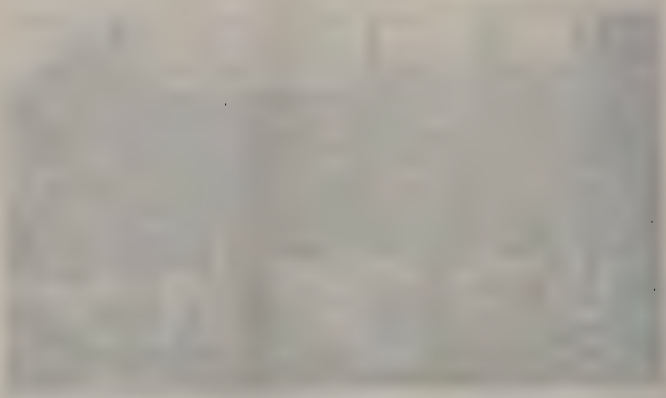
The City was given possession of the basement for 18 months, so as to facilitate the work of construction, and agreed to build new foundations capable of carrying a new building of eight stories or one of lighter construction not exceeding ten stories.

Fig. 3 is a plan of the building, and shows the positions of the east and west tunnels beneath the building and the proportion of the foundations cut away by them. The tops of these tunnels were just beneath the level of the footing stones under the building walls, as shown in Fig. 4.

An examination of the building was made before any work was done; its condition was carefully noted, and all defects or cracks were recorded. Survey marks were made at various places around the exterior walls, and their elevations taken.

FIG. 1.—GENERAL VIEW OF TRINITY VESTRY BUILDING AND ST. PAUL'S CHURCHYARD.

FIG. 2.—LONGITUDINAL LATTICE GIRDER BEING ERECTED AND FASTENED IN PLACE.



The following are the dimensions of the building:

Length on Church Street	164 ft. 0 in.
Width on Fulton Street	35 ft. 0 in.
Width on Vesey Street	48 ft. 0 in.
Length of exterior walls	428 ft. 0 in.
Area of ground plan	6 055 sq. ft.
Area of one floor (less walls)	5 340 sq. ft.

The following are the estimated weights:

Exterior walls	7 950 000 lb. =	3 975 tons.
Interior walls and partitions	994 000 " =	497 "
Floors	773 000 " =	386.5 "
Plastering, ceilings and walls	797 000 " =	398.5 "
Roof construction	393 000 " =	196.5 "
Equipment, trim, plumbing, etc.	501 000 " =	250.5 "
Live load, maximum	1 378 000 " =	689 "
Total weight on foundations,	12 786 000 lb. =	6 393 tons.

The area of the original foundations was:

Under exterior walls	2 140 sq. ft.
Under interior weight-bearing walls	400 " "
Total area of foundations	2 540 sq. ft.

The unit loading on the original foundations, assuming the weight to be distributed uniformly over the footings, therefore, was 2.51 tons per sq. ft.

The soil was sand, varying in places from fine to moderately coarse. Mean high water level was 23 ft. below the street, but the ground-water was found about 4 ft. 6 in. above that level. Pumping in the adjacent subway excavations reduced the ground-water to about high water.

After conferences, the City agreed that the new foundations should be able to support the building if four new stories of similar construction were added, or their equivalent in weight. These additional floors were estimated at 3 907 tons.

Original weight of building	6 393 tons.
Additional weight, agreed upon	3 907 "
Estimated load for new foundations	10 300 tons.

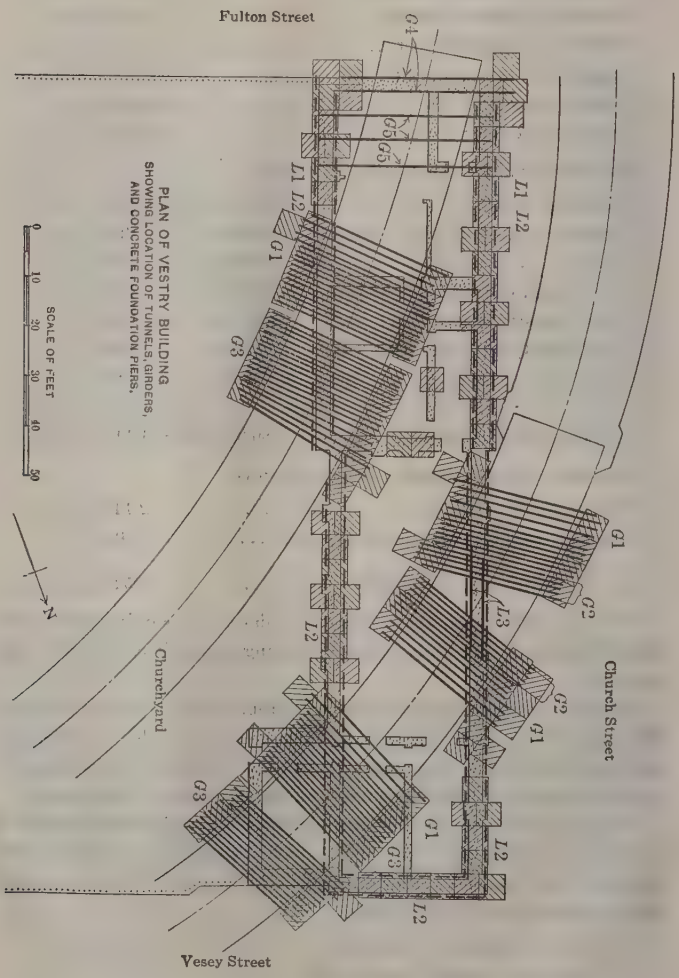

Fulton Street
Church Street
Churchyard
Vesey Street
G1
G2
G3
G4
G5
L1
L2
L3
PLAN OF VESTRY BUILDING
SHOWING LOCATION OF TUNNELS, GIRDERS,
AND CONCRETE FOUNDATION PIERS.
SCALE OF FEET
0
10
20
30
40
50
N

Fig. 3.

This estimated load on the exterior walls (10 300 tons divided by 428) equals 24 tons per lin. ft.

As the load would be supported principally by the side exterior walls, it was arbitrarily agreed that the underpinning under the side-walls should carry more weight and under the end-walls less weight than the average; or

End-walls	12	tons per lin. ft.
Side-walls.	28	" " " "

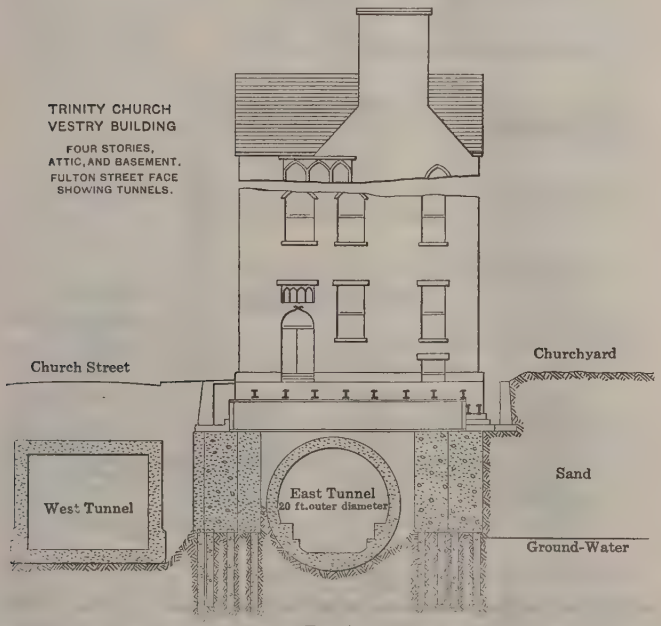


FIG. 4.

The plan for underpinning finally adopted consisted of sinking 115 pits, each 5 ft. square, under and around the building, but placed so as to clear the tunnels, as shown in Fig. 3. These pits were then filled with concrete. Girders were designed to span the tunnels, and were placed across the concrete piers in such a way that they would pass under the walls of the building. The load of the building was then

gradually transferred to these piers, or to the girders and by them to the piers. This plan will be better comprehended by referring to Fig. 4, which is a section at the Fulton Street face, and Fig. 5, which is a section through the building at about mid-length.

In Fig. 4, the west tunnel is under Church Street, clear of the building, and has reverted to the standard rectangular section. The east tunnel is just emerging from the building. It then changes to a

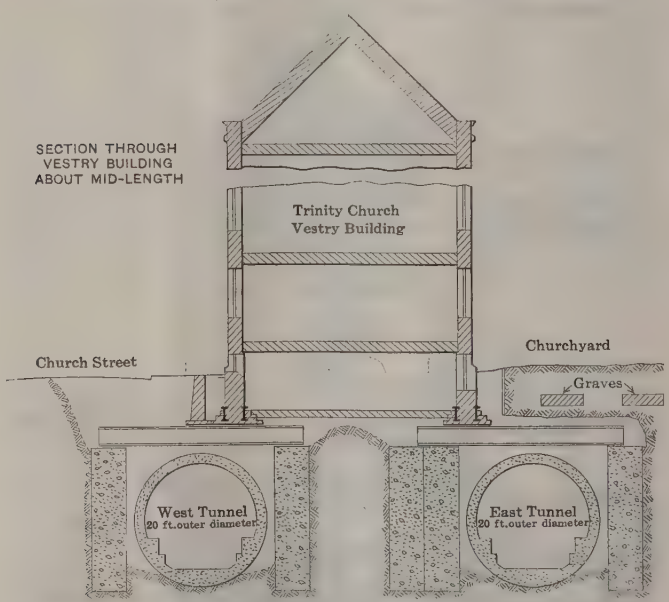


FIG. 5.

rectangular section, and unites with the west tunnel. In Fig. 5, both tunnels are under the building, or Churchyard, and are of circular section.

The first operation was to fasten 24-in. lattice girders on each side of the foundation walls. These girders are marked *L* 2, in Fig. 3, and are shown in detail in Fig. 6. At Fulton Street, the 24-in. girders were reduced to 12-in. so as to make room for the cross-girders marked *G* 5, in Fig. 3. These 12-in. lattice girders are marked *L* 1. The func-

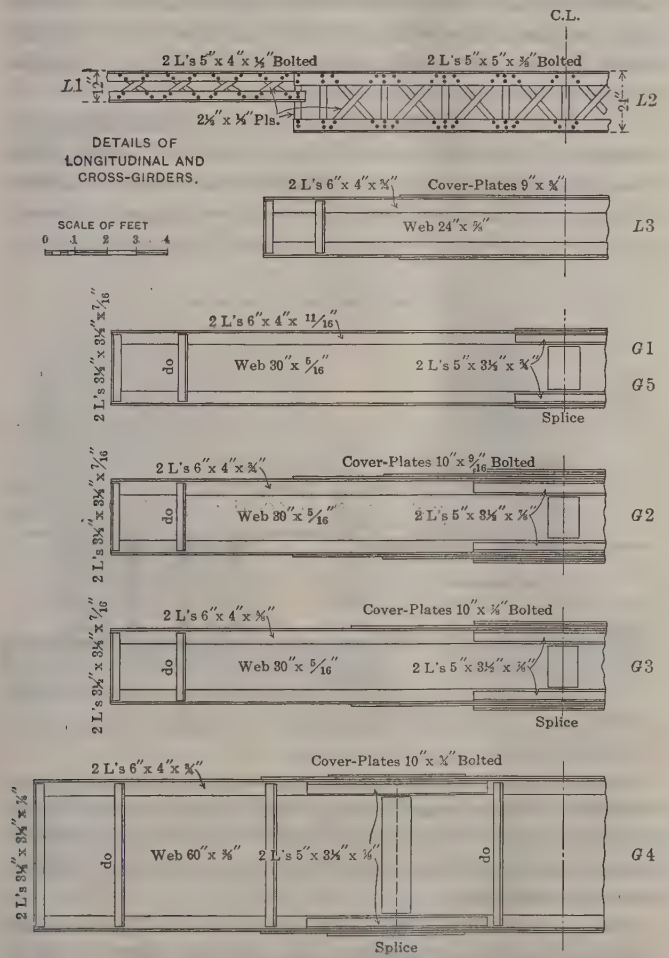

C.L.
2 L's 5" x 4" x ½" Bolted
2 L's 5" x 5" x ⅝" Bolted
L1
L2
2½" x ½" Pls.
DETAILS OF LONGITUDINAL AND CROSS-GIRDERS.
SCALE OF FEET
0 1 2 3 4
2 L's 6" x 4" x ¾"
Cover-Plates 9" x ¾"
Web 24" x ⅝"
L3
2 L's 6" x 4" x 11/16"
2 L's 3¾" x 3½" x 7/16"
do
Web 30" x 5/16"
2 L's 5" x 3½" x ¾"
G1
G5
Splice
Cover-Plates 10" x 9/16" Bolted
G2
2 L's 6" x 4" x ⅝"
Cover-Plates 10" x ½" Bolted
G3
2 L's 6" x 4" x ⅝"
Cover-Plates 10" x ¼" Bolted
Web 60" x ⅜"
2 L's 5" x 3¼" x ⅛"
G4

FIG. 6.

tion of these longitudinals was to strengthen and tie the brickwork together, that is, to make it more monolithic in character. The brick walls were thus enabled to distribute the loads from above over longer lengths, and to span safely the openings which were to be dug subsequently for the concrete underpinning piers.

At one place on the Church Street side, 24-in. longitudinal plate girders, 20 ft. long, were used in place of the 24-in. lattice girders. These are marked *L* 3, in Fig. 3. This change in design was made so that the load of the side-wall could be carried over a space left between nests of cross-girders, which had to be placed diverging, so that their

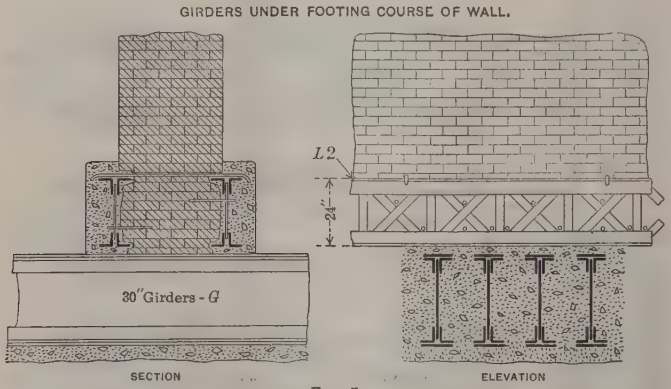


FIG. 7.

concrete piers could clear a foundation of an elevated railway column in Church Street.

Projections on the sides of the footing walls were cut away, leaving the vertical surfaces somewhat rough. This cutting was done by both hand-drills and by power channel drills. Grooves were cut for the angles of the top and bottom chords, in order that the girders could fit snug against the walls.

The girders were then doweled to the walls at frequent intervals. The dowels were square rods driven into the walls and bent over to catch the framing of the girders. Ties also were inserted in holes drilled through the wall at intervals. These bound the top chords of

the inside and outside girders together. The arrangement is illustrated in Figs. 7 and 8. After the girders were in place they were embedded in concrete, a strong bond being secured by the friction of the roughed surfaces of the sides of the walls aided by the dowels. To facilitate the placing of these girders, they were built up in place and bolted. Splices were staggered, both of the upper and lower chords and of parallel girders. Fig. 2 shows a 24-in. longitudinal lattice girder in process of erection. It is similar to those used under the Vestry Building.

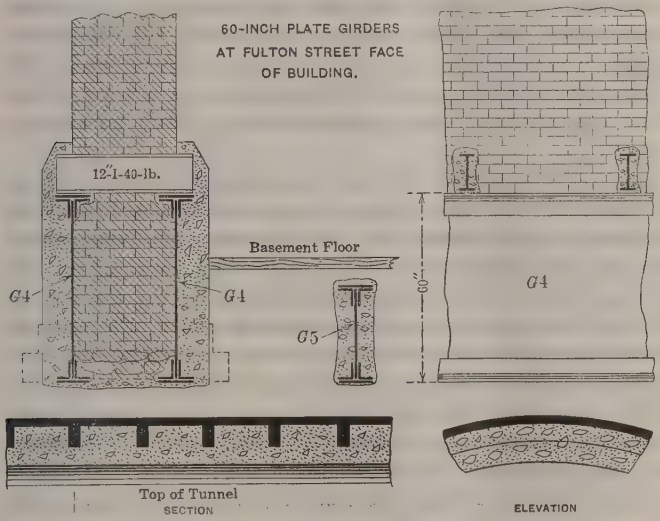


FIG. 8.

Across the Fulton Street face, two 60-in. plate girders were used in the same manner as the side lattice girders. These 60-in. girders are marked *G* 4, in Fig. 3, and were made in three parts for convenience in placing, the sections being bolted together when in position. They were tied together by 12-in. I-beams, 40 lb. per ft., as shown in Fig. 8, and were encased in concrete. The weight of the wall was transmitted to the girders by these needle I-beams, as it was not deemed wise to rely on the bond of the concrete casings.

After completing the work of fastening the longitudinal girders, *G* 4, *L* 1, *L* 2, and *L* 3, excavations for the underpinning piers were commenced. Excavations for adjacent piers were not made at the same time. When one pit was started, the next would be 30 or 40 ft. away, or as far as convenience would allow.

The pits were sunk by hand digging. As the excavations proceeded, the sides were box-sheathed with 2-in. planks laid horizontally. These planks were cut to fit, and when a set of four was in place the pieces were toe-nailed at the corners. When an excavation had advanced sufficiently, a 3 by 3-in. timber was put in each corner and spiked. Occasionally, these corner timbers were cross-braced with waling pieces which were wedged into place. No difficulty was experienced in this work. The average time required for three men to sink one of these pits varied from 3 to 6 days, including from 1 to 2 days in making the access.

Each pit was filled with concrete as soon as excavated. For piers directly under the walls, the spaces between the concrete tops and the footing course were packed with brick and cement, all tightly wedged, so that such pits could receive their share of the building load. For piers not under the walls, the plate girders, marked *G* 1, *G* 2, *G* 3, and *G* 5, in Figs. 3 and 6, were put into place and secured. The spaces between these girders and the footing course were then packed, and a portion of the load was transferred to the concrete piers, as before. These cross-girders were then encased in concrete, so as to protect the steel and make the nests of girders into solid floor-slabs. This feature is best understood by referring to Figs. 3 and 7.

As there were 115 of these concrete piers, each 5 ft. square, the average load on each was, for the present building (6 393 tons divided by 115 times 25) = 2.22 tons per sq. ft.; or, for the assumed future building, would be (10 300 tons divided by 115 times 25) = 3.58 tons per sq. ft.

In the Churchyard, some of the graves and vaults were in the way of the pit excavations. To avoid disturbing them, cross-tunnels were routed in from convenient sides, at sufficient depths to clear the graves, as shown in Fig. 5. When the correct locations were reached by the cross-tunnels, the pit excavations were carried down vertically in the usual manner.

It was the original intention to excavate the pits for the concrete piers to about ground-water level and then to drive concrete piles to a depth below sub-grade. As the subway pumping reduced the ground-water level, the plan was changed, the pits being carried down to about 1 ft. below sub-grade, and the piles omitted. However, at the Fulton Street end, short concrete piles were used under the pits, as seen in Fig. 4.

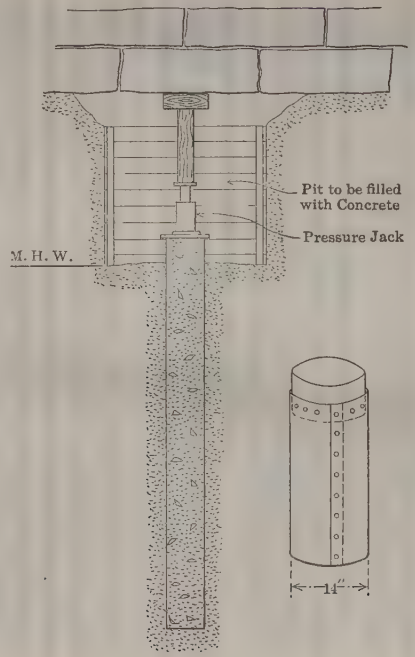
Elsewhere on this subway route some similar concrete piles were tested by the contractors, under the supervision of the writer. As these tests were interesting, the results are here given.

Two piles were tested under columns of the elevated railway in Trinity Place, between Rector and Thames Streets, during January, 1913, and three piles were tested under the retaining wall of Trinity Churchyard, just north of Rector Street, during February, 1913.

The piles were made by forcing into the sand a steel casing 14-in. in diameter, No. 12 gauge (about $\frac{1}{4}$ in. thick). The casing was made in sections about 24 in. in length, each section having an inside sleeve so as to permit one section to fit on top of the one just below, see Fig. 9. The steel casing was sunk by a pile-driver until its upper edge was level with the sand; then a new section was inserted, and the driving was continued until the proposed depth of pile was reached. The sand was removed by a small grab-bucket from the inside of the casing as the sections were sunk. When the entire casing was in place and the sand had been removed from the inside, the pile was formed by filling the casing with 1:2:4 concrete. For filling below ground-water, the concrete was put in paper bags and the bags were lowered into place and tamped. After about 2 ft. had been filled in this way the concrete was placed by a bottom self-dumping bucket.

The piles were tested, after the concrete had set, by a hydraulic jack, using the footing courses of the foundations as a base against which the jack was blocked. Owing to the short stroke of the jack, the pressure had to be relieved and the blocking readjusted as each pile sunk under the test loads. Whenever the pressure was relieved, there was a slight rebound of the pile. A graphic representation of Test IV is shown in Fig. 9, which illustrates this rebound.

Test I.—Steel casing, 9 in. outside diameter, driven inside a 14-in. casing which had buckled. The pile was 15 ft. long, and projected



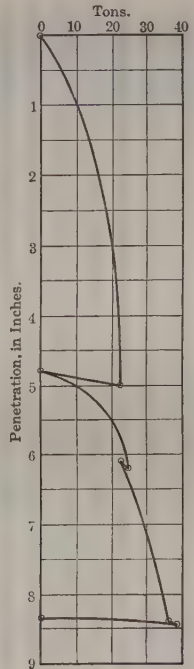


FIG. 9.

TEST OF
14 INCH CONCRETE PILE,
AND GRAPHIC RECORD OF TESTS.

Time.	Load, in Tons.	Penetration, in Inches	
		Each Test	Total
	0.0	0.0	0.0
	23.2	5.0	5.0
	0.0 +	0.250 ⊠	4.750
2:42	25.8	1.625	6.375
2:47	24.5	0.250 ⊠	6.125
2:48	32.0	1.375	7.500
2:53	30.2	0.0	7.500
2:54	36.5	0.875	8.375
3:04	38.3	0.062	8.437
3:05	0.0	0.125 ⊠	8.312

+ Pressure relieved, to reset blocking.

⊠ Rebound when pressure was relieved.

about 2 ft. below the 14-in. casing. The 9-in. casing was filled with concrete. The soil was fine, wet sand.

Time.	Test load, in tons.	Penetration, in inches.	Total movement from start of test, in inches.
3:59	0.0	0.0	0.0
4:00	0.63	0.0	0.0
4:06	18.80	0.276	0.276
4:16	21.40	0.012	0.288
4:18	31.40	0.228	0.516
4:28	31.40	0.156	0.672
4:33	31.40	0.024	0.696
4:37	37.70	0.864	1.560
4:42	37.70	0.324	1.884
4:45	0.0	0.336*	1.548

* Rebound when pressure was relieved.

Test II.—This test was made to measure the end resistance of a 9-in. outside diameter, pointed casing. A 14-in. steel casing, 12 ft. long, was first sunk, and about 2 ft. of sand allowed to remain at the bottom. Inside this casing a steel pile 9 in. in diameter, 14 ft. 6 in. long, of ½-in. steel, with cast-iron point, was driven until the point was below the outer casing. Neither casing was filled with concrete. The soil was fine, wet sand.

Time.	Test load, in tons.	Penetration, in inches.	Total movement from start of test, in inches.
3:15	0.0	0.0	0.0
3:16	6.3	0.875	0.875
3:18	12.6	4.0	4.875
3:24	0.0*	0.125†	4.750
3:26	12.6	0.8125	5.5625
3:29	14.4	1.1875	6.75
3:31	18.8	2.125	8.875
3:34	0.0*	0.25†	8.625
3:38	23.2	7.625	14.25
3:44	0.0*	0.1875†	14.0625
3:46	25.1	1.1875	15.25
3:51	25.1	1.0	16.25
3:52	0.0*	0.1875†	16.0625
3:57	31.4	1.8125	17.875
4:02	31.4	0.6875	18.5625
4:08	34.6	1.375	19.9375
4:06	0.0*	0.3125†	19.625
4:08	31.4	0.5625	20.1875
4:09	34.6	1.125	21.3125
4:10	34.6	0.75	22.0625
4:11	34.6	0.625	22.6875
4:12	0.0*	0.3125†	22.375

* Pressure relieved, to reset blocking under jack.
† Rebound, when pressure was relieved.

Test III.—Steel casing, 14 in. in diameter; length of pile, 18 ft.; concrete, 1:2:4 mixture, allowed to set for 10 days. The soil was fine, wet sand.

Time.	Test load, in tons.	Penetration, in inches.	Total movement from start of test, in inches.
4:10	0.0	0.0	0.0
4:11	1.9*	0.0	0.0
4:18	32.0		
	28.9 †	2.940	2.940
4:21	34.5		
	31.3 †	2.052‡	4.992

* Pile started to move when a load of 23 tons was reached.
† Load slowly reducing by a leak in pump.
‡ Rebound was not recorded.

Test IV.—Steel casing, 14 in. in diameter; length of pile, 12 ft. A concrete plug, 48 hours old, 13½ in. in diameter and 18 in. long, was first dropped into the casing. Concrete was poured into the casing above the plug and allowed to set for 9 days. The soil was fine, wet sand.

Time.	Test load, in tons.	Penetration, in inches.	Total movement from start of test, in inches.
....	0.0	0.0	0.0
....	23.2	5.0	5.0
....	0.0*	0.250†	4.750
2:42	25.8	1.625	6.375
2:47	24.5	0.250†	6.125
2:48	32.0	1.375	7.500
2:53	30.2	0.0	7.500
2:54	36.5	0.875	8.375
3:04	38.3	0.062	8.437
3:05	0.0	0.125†	8.312

* Pressure relieved, to reset blocking under jack.
† Rebound, when pressure was relieved.

Test V.—Steel casing, 14 in. in diameter; length of pile, 16 ft. 6 in.; concrete, 1:2½:4½ mixture, allowed to set 17 days.

Time.	Test load, in tons.	Penetration, in inches.	Total movement from start of test, in inches.
9:40	0.0	0.0	0.0
9:42	12.6	0.1250	0.1250
9:42:30	18.8	0.3750	0.5000
9:43:30	25.1	1.1250	1.6250
9:53:30	25.1	0.4375	2.0625

Test V.—(Continued.)

Time.	Test load, in tons.	Penetration, in inches.	Total movement from start of test, in inches.
9:54:30	31.4	0.9375	3.0000
10:04:30	31.4	0.2500	3.2500
10:06:00	35.8	1.1250	4.3750
10:08:00	0.0*	0.3750†	4.0000
10:17:00	6.3	0.0	4.0000
10:17:20	12.6	0.5000	4.5000
10:17:30	18.8	0.0	4.5000
10:17:40	25.1	0.0	4.5000
10:18:00	31.4	0.0625	4.5625
10:19:00	36.4	0.8135	5.3750
10:19:20	37.7	0.3750	5.7500
10:24:20	37.7	0.3750	6.1250
10:29:20	37.7	0.1250	6.2500
10:34:20	37.7	0.0	6.2500
10:49:20	37.7	0.0	6.2500
10:50:20	0.0	0.3125†	5.9375

* Pressure relieved, to reset blocking under jack.
† Rebound, when pressure was relieved.

Some conclusions, deduced from a study of the test loads and penetrations, are:

(1) The initial loads, which the piles supported before sinking, were variable.

(2) The sinking increases more rapidly than the load.

(3) When the pressure is relieved, the pile rebounds.

(4) When the same load is again put on the pile, the penetration is greater than the original penetration, that is, the pile sinks more than its rebound.

(5) The supporting power of one of these concrete piles depends chiefly on the area of its base. The side friction in sand probably does not exceed from 5 to 8% of the load.

(6) To secure the maximum supporting power without additional settlement, a concrete pile should receive its permanent load while under pressure, that is, the permanent underpinning load should be transferred to it without relieving the test load.

(7) This latter condition can be accomplished by placing struts between the footing to be underpinned and the pile top before the pressure of the jack is relieved.

After the underpinning work had been completed, and the load of the Vestry Building had been transferred to its new foundations, excavation beneath the basement was commenced, and the tunnels were

allowed to advance toward the building. At the same time, excavation under Church Street, which had been decked with wood, also was being made.

The tunneling commenced from a shaft in Vesey Street, and the east tunnel was kept somewhat in advance of its westerly mate. Both tunnels were worked forward from Vesey Street across the Churchyard toward Fulton Street. They are 20 ft. in outside diameter, and are lined with cast-iron plates covered with concrete inside.

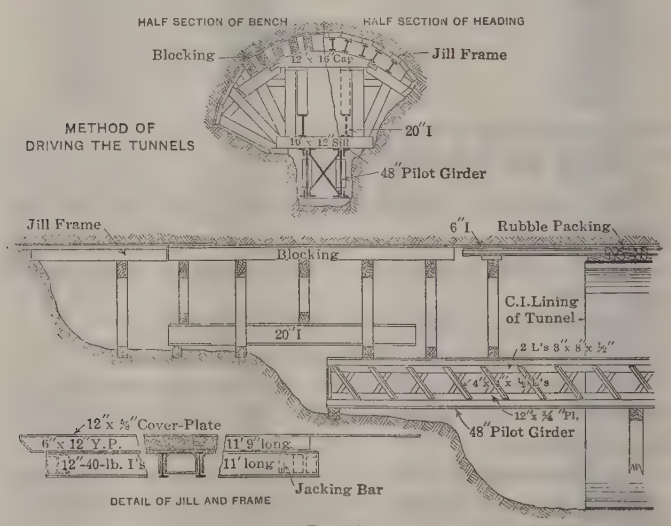


FIG. 10.

As the soil was sand, varying from fine to medium coarse, the method of tunneling adopted by the contractors is shown in Fig. 10. Jills, arranged side by side in the curve of the roof, and supported by cap timbering, were driven forward by jacks. As the jills were pushed ahead, the space behind them was sheeted and filled with blocking resting on the caps. The sides of the tunnel were sheeted with 2-in. planks, held in place by braces and struts. A steel pilot girder formed the base from which the various braces and capping were supported. This pilot girder also formed the truss for supporting the centering machine which raised and put the cast-iron lining sections in place.

The excavation was carried forward by an advance drift close under the roof, and benches formed by deeper excavations to sub-grade were kept back at safe distances, depending on circumstances. Timbering was placed as the section was enlarged, and the cast-iron lining plates of the tunnel were erected as soon as the full section was cut. Fig. 11 shows one of the headings as it advanced under the Churchyard.

The jills were made of timber, sheathed with steel plates bent over the forward nose to form a cutting edge. These jills rested on, and were bolted to, steel frames, built up as seen in Fig. 10, and were all supported on timber capping braced radially from the tunnel center. They were pushed ahead by hydraulic jacks, the center one being kept slightly in advance of the others. The sand was removed in cars to the Vesey Street shaft and hoisted to the street.

The tunneling was accomplished without serious accident, although the method required great care and attention to prevent slips. There was a noticeable loss of sand, that could not be avoided, and there was a slide of sand at one place which caused a marked settlement of the Churchyard. The engineers of the Public Service Commission approved the tunneling plans before work started, and the contractors are entitled to credit for the manner in which the work was done. The tunneling beneath the Churchyard was accurately carried forward, and the headings entered the spaces between the concrete foundation piers as planned.

When the drift reached the partial excavation beneath the building, the advancing jills, as they emerged from the sand, are shown in Fig. 12.

Fig. 13 shows the excavation beneath the building. The concrete foundation piers are seen on both right and left, and the concreted cross-girders are shown overhead. The cast-iron lining of the tunnel can be seen in the background. As the lining plates were erected, the spaces around them were back-filled.

Fig. 14 is another view of the excavation beneath the building. The basement windows can be seen through an opening between the concreted cross-girders.

Fig. 15 shows the east tunnel at Fulton Street, with the concreted cross-girders, *G* 5, overhead. At the top of the picture in the foreground is seen the footing course of the Fulton Street wall of the building.

Fig. 16 is a view inside the completed cast-iron lining of one of the tunnels. It shows the concrete key of the inside lining which was jacked into place in advance of the general lining.

The work of underpinning and of transferring the load to the new piers caused a settlement of the building, as shown by the surveys, as follows:

At N.E. Corner, on Vesey Street	$1\frac{11}{16}$ in.
" N.W. " " " "	$1\frac{5}{16}$ "
" center of Churchyard side	$1\frac{13}{16}$ "
" " " Church Street side	$2\frac{5}{16}$ "
" S.E. Corner, on Fulton Street	$2\frac{13}{16}$ "
" S.W. " " " "	$2\frac{3}{8}$ "

A few cracks of minor importance developed, and the plaster spalled in some places.

As shown by the survey figures, the settlement was very uniform. This even settlement, and the slight damage to the building, resulted from digging the pits without regularity, that is, by not excavating them from one end of the building toward the other.

The contract price for this section of the subway, extending from a point on Broadway, just north of Barclay Street, to Church Street, a distance of 1 030 ft., was $982 740.

The work of underpinning was commenced in June, 1913. The longitudinal girders along the wall footings and the cross-girders were all placed by February, 1914. Both tunnels were completed beneath the building by June, 1914. The back-filling under the building was completed and the basement floor and rooms restored by October, 1914, or 17 months after the active work of underpinning was started. The final restoration of the property was delayed by the work under Church Street, requiring the adjustment of street steam-supply pipes, telephone ducts, sewers, etc., until April, 1916.

The contractors were Frederick L. Cranford, Inc., and the work was done under the personal direction of Mr. F. L. Cranford and J. C. Meem, M. Am. Soc. C. E.; the Superintendent on the work was Mr. H. L. Robinson, and the Assistant Engineers were Messrs. H. P. Moran, in charge of the office, and W. McI. Wolfe, Assistant Engineer on field work. For the Public Service Commission, Alfred Craven,

FIG. 11.—VIEW IN ADVANCE HEADING OF TUNNEL UNDER ST. PAUL'S CHURCHYARD.

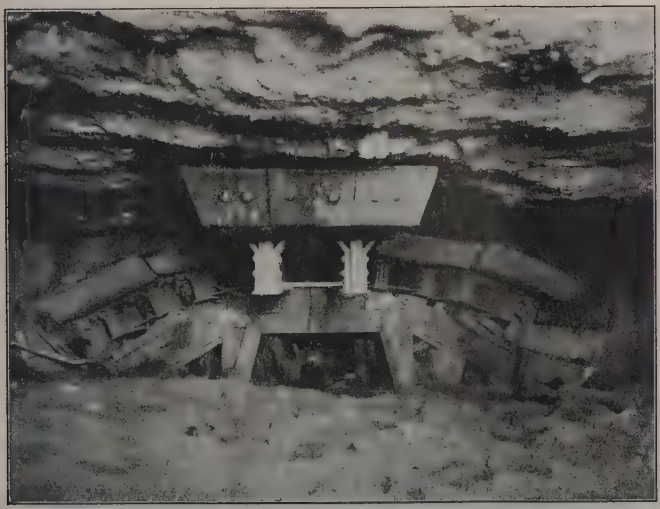


FIG. 12.—TUNNEL JILLS BREAKING INTO SPACE UNDER TRINITY VESTRY BUILDING.

Fig. 13.—View Under Trinity Vestry Building, with the Concrete Piers on Both Right and Left.

Fig. 14.—View Beneath Building, Showing Basement Windows Between Concreted Cross-Girders.

FIG. 15.—EAST TUNNEL AT FULTON STREET. FOOTING COURSE OF BUILDING IS SHOWN OVERHEAD.

FIG. 16.—VIEW SHOWING CONCRETE KEY ON INSIDE OF COMPLETED CAST-IRON LINING.

M. Am. Soc. C. E., was Chief Engineer; Robert Ridgway, M. Am. Soc. C. E., Engineer of Subway Construction; and Mr. Jesse O. Shipman, Division Engineer. The writer was Consulting Engineer for the Corporation of Trinity Church, and was assisted in supervising the work by D. C. Johnson, Assoc. M. Am. Soc. C. E., and Mr. W. E. W. Moore. The photographs were taken by Mr. Edwin Levick.

DISCUSSION

Mr. Meem.

JAMES C. MEEM,* M. AM. SOC. C. E. (by letter).—The author has covered the subject of this paper so ably and fully that very little remains to be said, except in reference to the elaboration of certain points which he has mentioned.

The sectional roof shield method of tunneling has been used on many important pieces of work, and is doubtless familiar to many, but the use of the "pilot-girder" in connection therewith is new. This was made of two lattice-girders, 4 ft. high, cross-tied and braced to form a continuous box-girder, the rear members, as stated, being carried forward and placed in position as required. The outer chords were made longer than the inner ones, in order to accommodate it to the curvature of the tunnel. The office of this girder was to carry the falsework over the bottom excavation between the heading and the completed section of the tunnel, and it proved to be a very satisfactory and efficient substitute for the arched timber method which had generally been used in connection with the sectional shield.

Owing to the shortness of the radius of curvature of this tunnel and the extreme accuracy with which the rings had to be set (the contractor was limited absolutely to a variation of not more than 2 in. at any point), it is doubtful whether any other method of tunneling could have been used so successfully and advantageously.

In the matter of the concrete key in the roof, which is shown in Fig. 16, very satisfactory results were obtained by filling beforehand these spaces in the cast-iron ring above the key. Heretofore, as far as the writer knows, no satisfactory method of filling these spaces after placing the key has been found, except by grouting. This key was placed by using a form with a movable bottom; on this the concrete was placed and then the whole mass was jacked into the roof space and maintained until it had set.

In the matter of the very limited dependence to be placed on skin friction resistance of piles, as noted by the author, the writer agrees very heartily. He believes that skin friction resistance is often mixed up with bearing resistance, and that, as a factor by itself, it has very little value, especially with smooth-bore piles. To assume a value of, for instance, 1 000 lb. per sq. ft. for skin friction resistance, as many engineers often do, is to assume that the pile withstands an external pressure of approximately 2 500 lb. per sq. ft. It is improbable, however, except in cases of very long piles in deep fluid strata, that piles are ever subjected to a pressure of more than a few pounds per square foot, and if this is true, skin friction resistance, of course, can be only a fraction of this pressure, so that, as stated, it may be neglected

* Brooklyn, N. Y.

in nearly all cases. The pile, undoubtedly, derives its real support from the so-called "bulb of pressure" at its base. Mr. Meem.

ELIAS CAHN,* ASSOC. M. AM. SOC. C. E. (by letter).—The writer has read this paper with great interest, particularly the part relating to the tests on concrete piles, and considers the Profession greatly indebted to the author for their publication. It is a pity, however, that these valuable tests were not extended to include the influence of other factors bearing on the supporting power of these piles and their settlement under load, no less important than those considered. Time is one of these factors, and should not be neglected in dealing with earth, because stresses travel so slowly through this medium. All the tests, particularly No. 5, give indications of this, but the time taken in any of them, about 1 hour, is very far from sufficient to develop this influence fully. The depth to which a pile is driven is also an important factor. Mr. Cahn.

The following, therefore, are suggested as additions to the author's conclusions:

(8) The settlement accompanying the development of a certain pressure will be greater, the longer the time taken to develop this pressure.

(9) A pile will settle under the action of a constant pressure at a rate and for a time depending on the pressure, the area of the base of the pile, the kind of material which the pile penetrates, and the surcharge over the horizontal plane through the bottom of the pile.

The earth immediately under the pile, when subjected to the maximum pressure of the jack, as is clearly indicated by the rebound, is in a state of excessive compression, which undoubtedly decreases in all directions from this point. There will be a tendency to equalize this compression in time, thereby reducing that immediately under the pile, and either decreasing its resistance or causing further settlement.

The conditions under which equilibrium is finally obtained and the total settlement in time under a definite pressure or load, are the determinants of what the safe load should be, rather than the pressure it is possible to develop with the jack in a short time.

In Conclusion (5) no mention is made of the effect, on its supporting power, of the depth to which the pile is driven. This depth, for any given earth, determines the compression that the material under the pile will stand permanently, and therefore the load it will support.

The practice of considering the safe load as one-half the maximum pressure obtained with the jack, therefore, is not logical unless the depth of penetration and the kind of earth as well as the area of the

* New York City.

Mr. Cahn. base of the pile are considered in determining the pressure to be obtained with the jack. Aside from any difference in the skin friction, a pile driven 30 ft. into the earth would have a greater supporting power than if driven only 15 ft., though the same pressure is obtained with a jack in the manner described in the paper, simply because the earth at the lower level will stand permanently a greater degree of compression than at the upper level.

The author has done more than his share in conducting the tests recorded, and it is to be hoped that those who may have the opportunity in the future will continue where he has left off, clearing up this matter of the effect of time and of depth of penetration on the settlement of a pile under a constant pressure after the earth under it has been compressed by a far greater pressure.

Mr. Thomson. T. KENNARD THOMSON,* M. AM. SOC. C. E.—It is a pleasure to see a well-designed piece of work well carried out, and then well described; and the Society is once more indebted to Mr. Parsons.

Whenever the writer sees a "jack" under a building he thinks of the comments of an eminent lawyer who was wont to laugh at engineers for their careless use of the English language, for he claimed that they nearly always referred to the process of "jacking up the building"; whereas, they did not do anything of the sort, as the method was to put a jack between the base of a building and a post or sill resting on the ground and then, by operating the jack, force the post or sill into the ground, keeping the building itself exactly where it was, instead of "jacking it up".

Sentimental reasons against disturbing the church building and graveyard undoubtedly entered into the selection of this design. The contract price, as stated by the author, was $982 740 for 1 000 ft. Otherwise, it might have been cheaper to tear down the old building instead of underpinning it. However, the occupants of the building and countless thousands of pedestrians gained by the lack of disturbance of surface conditions.

It was fortunate in this case that two of the most dangerous elements, often encountered in underpinning operations, were absent—that is, water, and bad design or construction of the building to be underpinned.

The speaker recalls one building which caused him much worry, due to its very defective—to say the least—construction. It was some twelve stories high; the two lower floors were supported by cast-iron and steel columns of various shapes and the floors above were carried by brick walls. The first joint uncovered disclosed the fact that two 20-in. beams did not rest on the shelf brackets of the cast-iron columns, as there was at least a 2-in. space between the bottom of the I-beams

* New York City.

and the top of the shelf, and although there had been provision for several bolts to connect the flange of the columns to the webs of the beams, only one bolt, for each beam, had been put in place—so all the support which the heavy 20-in. I-beams had was one ¾-in. bolt each. If a safe or any other heavy article had ever been moved over these beams, there would have undoubtedly been a bad collapse. Mr. Thomson.

Needless to say, the disclosure of such an unsafe condition in the first joint uncovered caused much worry about the possible condition of the many joints in the building, which it was not possible to uncover for examination. However, the building, which rested on quicksand, at the surface of the ground-water, was safely underpinned by forcing cylindrical caissons to bed-rock from 60 to 70 ft. below the water surface, through New York quicksand.

The speaker has underpinned walls eighteen stories high, by putting cylindrical caissons down by the Breuchaud method, through 70 ft. of water and quicksand, and then constructing a cellar for the new building 35 ft. under water.*

In underpinning a six-story building by jacking down 16-in. pipes until they were supposed to have reached hardpan (as a matter of fact they offered so much resistance to the jacks that the weight of the old building was really taken off the old foundations, as shown by horizontal cracks in the brickwork), it was found, after all the sand had been washed out of the cylinders and they had been filled with good concrete, which should have made the underpinning strong enough to carry very much heavier loads than those to which they were subjected (for the building was practically raised by jacking against the empty pipes, that is, before they had been filled with concrete) that the cylinders actually settled when the pneumatic caissons were being sunk near-by, thus loosening up the soil around the pipes and thereby destroying the friction. It would seem, therefore, that dependence cannot be placed on such pipes when the surrounding material is likely to be disturbed.

The author states that in the Trinity Vestry Building the friction of the material around the pipes was disregarded, and that the pipes were treated as short columns having all the load transmitted to the base, but these pipes were comparatively short. The length of such cylinders is often 30 ft. or more; they are too long to be considered as short columns, and offer very considerable resistance, due to the friction of the earth.

Owing to this lack of reliability, the speaker does not usually recommend underpinning with cylinders, unless they are large enough to permit a man to enter them to excavate the material and then fill them with concrete. A cylinder having an outside diameter of 30 in. is about the smallest that can be used for this purpose.

*A full description of this work was published in *Engineering News*, March 28th, 1901.

Mr. Thomson.

It pays to use an ample thickness of metal in the cylinders, for the speaker has seen them made so thin that it was impossible to see the bottom of the excavation from the top, owing to the warping of the pipes while being jacked down.

On removing some old buildings on Wall Street, New York City, which, about 14 years previously, had been underpinned by 14-in. pipes jacked down and filled with the best of concrete (paper bags having been used there also), except for the bottom 2 ft. of the cylinder, it was found that it had been impossible to pump out all the sand, and the concrete had probably set without thoroughly compressing the sand in the lower 2 ft. of the pipe.

This, of course, would allow the building to settle at least a few inches, if it ever happened that the friction of the soil against the pipe was insufficient to carry the load, as is often the case when the soil is disturbed.

In underpinning, as in other foundation work, no hard-and-fast rules can be laid down, as each building requires a treatment of its own.

The author refers to the rebound of the cylinders after the jacks were removed, and the method taken to prevent it. The usual method, however, is to use steel wedges, either between steel plates and beams, or between granite wedging blocks.

The speaker tested some open concrete caissons for a bridge in New Jersey a few years ago, and measured the rebound. The cylinders were of reinforced concrete 6 ft. 6 in. in diameter, having an open well, about 3 ft. 6 in. in diameter from the bottom to the top, to permit the material to be excavated by clam-shell buckets.

After the caisson had penetrated some 75 ft., and was resting on what was supposed to be fairly good sand, it was filled solid with concrete to the top, which was 87 ft. from the cutting edge, making a solid concrete pier, 6 ft. 6 in. in diameter, from 12 ft. above ground to the cutting edge.

After this had stood for some time, a load of steel rails was gradually applied until a maximum of 10.24 tons per sq. ft. on the base of the pier was attained.

A gradual settlement of $\frac{5}{8}$ in. resulted, and, when the load was removed, the rebound was $\frac{1}{4}$ in., leaving a supposed permanent settlement of $\frac{3}{8}$ in. As this settlement had been caused by applying 10.24 tons per sq. ft., and as the total weight of the bridge, with its live load, would not exceed 6 tons per sq. ft., it would seem that the piers would be perfectly safe. As a matter of fact, however, the jar of the trains started fresh settlement in a number of the piers, and this continued for about one year, when the maximum of 6 in. was reached. This was about 5 years ago; and since then there has been no trouble.

Strange to say, the piers which settled were those which were supposed to rest on the best material. Mr. Thomson.

The speaker once constructed a tunnel 7 ft. in diameter, by the poling-board method, under Cedar Street, New York City, and then ran a branch tunnel 4 ft. in diameter for 104 ft. up the street. The 7-ft. tunnel was just below the sewers and other pipes, and just above the water line at the bottom, as about 6 in. of very damp sand had to be handled. The work was done so carefully that there was absolutely no settlement in the street, and, as a matter of fact, nobody except those concerned in the construction knew that the tunnel had been built.

It might be interesting to mention the fact that in Michigan, where there are mining shafts 4 000 ft. or more in depth, it has been noted that heavy weights dropped from the top of the shaft never reach the bottom, as the revolution of the earth causes them to strike the side (always the same side) of the shaft, and become lodged there. This was first noticed when a very heavy weight was accidentally dropped from the top and it was feared that the men in the shaft must have been killed, but the men did not even know that anything had fallen, because it never reached the bottom.

JAMES F. FOUHY,* ESQ.—The speaker would like to know Mr. Parsons' opinion as to whether or not the rebound of the piles was due to looseness at the joints of the pile sections. Such rebound can hardly be attributed to loose joints when the piles are concreted. The speaker has tested numerous piles recently before placing the concrete filling. The sections were from 12 to 16 in. in diameter, $\frac{5}{16}$ in. thick, and from 7 to 12 ft. long. Tests up to 40 tons were made with piles in clay, and 80 tons with piles on rock. In all the tests, the rebound was noted, and it is not unlikely that play at the joints was responsible therefor. Mr. Fouhy.

CHARLES RUFUS HARTE,† M. AM. SOC. C. E.—The speaker would like to know whether the rebound occurred immediately after the pressure was removed, or whether it was some time afterward that the piles showed a tendency to come up? Mr. Harte.

In connecting the Providence Division of the New York, New Haven and Hartford Railroad with the South Station, at Boston, Mass., in 1889, the necessity of maintaining traffic on the important streets crossed led to the use of a pile-driver with very short leaders, and sectional piles, which were made by sawing ordinary piles into 10-ft. lengths and fitting the lower end of each piece with a wrought-iron ferrule about 12 in. long, and a central dowel of 2-in. pipe. A special driving head, consisting of a heavy ferrule with a short section

* New York City.
† New Haven, Conn.

Mr. Harte. of pile, received the direct blow of the drop-hammer and protected the upper end of each section; as soon as a section was flush with the ground, this head was removed, a new section fitted on it, the head put on the upper end, and the driving continued until the pile was seated in the hard clay from 40 to 60 ft. below the surface. The material overlying the clay was salt marsh mud stiffened by the ashes, earth, and other filling material with which the Back Bay district had been reclaimed; it flowed laterally and vertically as displaced by the piles, forming a mound in some cases 2 or 3 ft. higher than the original surface, and, through the skin friction, opened the joints of the piles already driven, causing the tops to come up in some cases nearly 1 ft.

This adjustment of pressure extended over some little time, but, as a rule, if the piling of a group was tapped by the driver 48 hours after the last pile was driven, there was no further movement.

With sand, however, as in Mr. Parsons' case, the speaker would not expect such delayed action. The Boston fill had decided elasticity, which here would be lacking.

Mr. Connelly. Joseph A. A. Connelly,* Assoc. M. Am. Soc. C. E.—If the speaker understood Mr. Parsons correctly, he has stated that the load used in testing the piles was from 40 to 50% in excess of the required load. He has also stated that the pile was jacked down under this excess load, was held in place by the jack, and that the load of the superstructure was then wedged hard down on the pile, so as to prevent any motion. The speaker presumes the motion referred to was the rebound of the pile.

Assuming the required load on the pile to be 40 tons, the load under which the pile would be jacked down and held in place would be 60 tons. The speaker would like to know how a pile under a pressure of 60 tons can be held in place by a superstructure load of 40 tons. It is very probable that even 40 tons is in excess of the load which the pile actually receives from the building, while the 60 tons is actually placed on the pile by the jack.

It has been suggested that the rebound may be due to motion at the joints of the pile, but, in the speaker's opinion, this is not the case. On work of which he has had charge, steel casings, filled with concrete, were driven to a depth of 34 ft. These casings were made up of seventeen 2-ft. sections, and thus there were sixteen joints in each pile. As in the cases mentioned by the author, the piles rebounded about $\frac{1}{4}$ in. when the jacks were released. If all this rebound was due to motion at the joints, and was considered to be equally divided, the movement at each joint would be $\frac{1}{64}$ in.; this, in the speaker's opinion, is too small to lend much support to the theory that pile rebound is due to joint motion.

* New York City.

A statement has also been made that a pressure of 77 tons was sustained by an empty pile made up of short steel sections; this pile resistance, under conditions such as those outlined in the paper, seems to be high; one would be led to suspect that an obstruction had been encountered. Mr. Connelly.

J. S. Branne,* M. Am. Soc. C. E.—The speaker wishes to ask Mr. Parsons' opinion as to the behavior of the steel-cased concrete piles. Mr. Branne.

It is stated in the paper that the piles rebounded after releasing the pressure exerted by the jack which had forced them down. It seems to the speaker that this rebound is due, in a large measure, to lack of skin friction in these piles—the surface being a smooth steel plate—and that this would not occur in piles with a rough surface, as in a concrete pile cast and seasoned before driving, or even in a wooden pile, the latter having more or less knots and irregularities. The material through which the piles were forced is stated to have been coarse sand, and this should have offered quite some resistance to rebound. The speaker believes that, in a soft clay soil, there would be a rebound, due to the elasticity of the soil and the lubricating influence of the moisture in the clay.

Referring to the jill frames which were driven forward with jacks, the speaker would like to know how large a section of frame was pushed forward at one time, that is, how wide a section; and also how much pressure was required to push such section forward? In case of an unusual obstruction, as, for example, a large boulder, the speaker supposes that such obstruction would be removed by the laborers with picks and shovels.

A. W. Buel,* M. Am. Soc. C. E.—The speaker would like to know whether it is possible that a part of the rebound is due to the elastic recovery of the pile itself? Mr. Buel.

With a modulus of elasticity, E_c, taken at 2 000 000, the elastic recovery of a pile, 14 in. in diameter and 40 ft. long, under a load of 40 tons, would be 0.01 ft., or $\frac{1}{8}$ in.

$L = 40$ ft.;
Diameter $= 14$ in.;
Area $= 154$ sq. in., nearly; and
$P = 40$ tons $= 80\,000$ lb.

Then, if E_c is assumed at the usual value of 2 000 000, there will result:

$$f = \frac{P}{A} = \frac{80\,000}{154} = 520 \text{ lb. per sq. in.}$$

$$\text{Deformation} = \text{elastic recovery (nearly)} = e = \frac{f\,L}{E_c}$$

$$= \frac{520 \text{ lb.} \times 40 \text{ ft.}}{2\,000\,000} = 0.0104 \text{ ft., or } \tfrac{1}{8} \text{ in.}$$

* New York City.

Mr. Buel. The average final rebound, of the four tests in which it was recorded, appears to have been 0.2715 in., or slightly more than ¼ in. The elastic recovery of the pile itself is shown to be just about one-half of this, if the modulus of elasticity is taken at 2 000 000. Considering the conditions under which the concrete is placed, it would not be surprising if the modulus were less than 2 000 000, in which case the elastic recovery of the pile itself would be a still larger proportion of the total final "rebound". The remainder of the final rebound—probably not more than ⅛ in., and possibly less—might be just as well described and explained as the "elastic recovery" of the material supporting the pile, as to call it a "readjustment of the particles". In the last analysis, there may not be much distinction between the two descriptions or explanations, but if the phenomena can be explained by the action of well understood properties of matter, there would not seem to be any good reason for seeking new theories.

If the figures given by the speaker are correct, and the rebound is due, in about equal parts, to the recovery of the pile and of the material supporting it, then both would be of about equal importance. The records of the five tests indicate stress-strain relations not very different from those characteristic of concrete columns. This would naturally be expected, as both the concrete and the supporting material are imperfectly elastic and, like the concrete, the supporting material probably has a variable modulus of elasticity.

Mr. Parsons. H. de B. Parsons,* M. Am. Soc. C. E. (by letter).—The writer will endeavor to make clear some of the points raised in the discussion.

Replying to Mr. Harte: The rebound of a concrete pile occurred as soon as the pressure was removed. In other words, the top of the pile rose while the hydraulic pressure was being relieved from the jack. When all the pressure was relieved, the rebound ceased.

Replying to Mr. Connelly: Concrete piles used in subway underpinning work, as described in the paper, are designed to carry safe loads, but are tested to loads in excess of the permanent safe loads. Thus, a building pier might support a load of 200 tons. If each pile were intended to carry 40 tons, there would have to be at least five piles beneath the pier. In such a case, each pile could be safely tested to a load in excess of 40 tons and be held in place under such a load. When the last pile was tested, it would temporarily relieve the excess loads from the others.

Although there is nothing in the paper which refers to a pressure of 77 tons on an empty pile made up of short steel sections, pressures as great as this are sometimes developed in sinking the empty steel casings.

Replying to Mr. Branne: Each jill-frame consisted of three 6 by 12-in. timbers, making the wooden part about 36 in. in width, sup-

* New York City.

ported on an iron frame composed of two 12-in., 40-lb. I-beams, about 18 in. centers, as shown in Fig. 10. Only one jill-frame was pushed forward at a time, and the center frame was kept slightly in advance of the side frames. The pressure used in moving a frame naturally varied owing to local resistances. The frames were moved by 3½-in., hydraulic rams, and the average pressure to start a frame was about 5 000 lb. per sq. in., but this fell, as the frame moved, to about 3 000 lb. per sq. in. Mr. Parsons.

Referring in general to the remarks on the rebound of the concrete piles: It can be stated that the rebound was not attributable in any way to looseness of the joints of the steel casings. The function of the steel casings was to hold back the sand and to act as a form for the concrete. The loading was placed on the concrete and not on the casing.

The rebound may be due in part to the elastic recovery of the concrete, as mentioned by Mr. Buel, although his estimate for the modulus of elasticity is probably somewhat low. The rebound apparently was caused by the particles of sand under and around the foot of the pile readjusting themselves as the pressure on the pile was reduced. This "elastic recovery" of the sand and the "elastic recovery" of the concrete are probably both intimately connected with the rebound, although with present knowledge it is not possible to tell how much is due to the one and how much to the other.

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Paper No. 1381

SURGES IN AN OPEN CANAL

By R. D. Johnson, Esq.

With Discussion by Messrs. Karl R. Kennison and Irving P. Church.

Synopsis.

This paper points out a rational theory on which to base research into the question of the rise of water in a canal following an interruption of flow, due, for example, to a shut-down of a water-power plant; it calls attention to the analogy between this surge and the phenomenon known as the "hydraulic jump".

This interesting subject always comes up in connection with the problem of how high to build fore-bay walls to avoid overflow in case the motion of the water in a canal is suddenly arrested by a short circuit. Trouble from this source seems to be very infrequent, and yet a sound theory for the computation of the height of the surge wave has never come to the writer's attention. The reason that wash-overs have not been more common seems to be due to the fact that, for ordinary velocities, the surge is comparatively small, and a good fair guess usually proves a sufficient safeguard; nevertheless, it may be interesting to set forth what appears to be the beginning of a sound theory, applicable to such cases.

Neglecting friction, in a smooth, rectangular flume, the sudden dropping of a gate would seem to cause a backward rolling wave which consumes a part of the energy of the oncoming water in eddy losses, and accounts for the remainder in an increased depth behind the wave,

the water standing still and level between the wave and the gate. On this theory, an equation from which the depth of the water may be determined is expressible through recourse to the well-known law that force is equal to the rate of change of momentum; for, if the depth of the water in motion is d and that of the water at rest is D, the total free force acting (for unit weight of fluid and unit width of flume) is $\frac{D^2 - d^2}{2}$ and, in the time, t, during which Qt cubic feet of water passes, with a velocity, v, and also Qt cubic feet of water is projected backward over the top, so to speak, of the oncoming stream, the quantity of water brought to rest is Dt multiplied by the velocity of propagation of the wave, or,

$$Dtv_3 = Dt \times \frac{Q}{D - d},$$

the change of momentum is

$$\frac{DQtv}{g\,(D - d)},$$

and the rate of change of momentum is

$$\frac{DQv}{g\,(D - d)},$$

whence,

$$\frac{D^2 - d^2}{2} = \frac{D\,d\,v^2}{(D - d)\,g} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

from which D may easily be determined by trial.

It may be observed that as velocity is only relative, the height of the "jump" which takes place in this case should agree exactly with the formula for the "hydraulic jump", if the proper corrections are made in the velocities relative to the earth, in such manner that the wave would "stand still" in the ordinary acceptation of the term. In other words, no error in theory is introduced if, while the above phenomenon is in progress, the whole flume is regarded as moving, bodily, with a velocity, relative to the earth, equal and opposite to that of the wave propagation; and such modifications ought to, and do, reveal the formula for the ordinary "hydraulic jump".

In this case, the absolute velocity of the water approaching the wave would be

$$v + \frac{Q}{D - d} = v_2$$

and the absolute velocity of the deeper water, at depth, D, would be

$$\frac{Q}{D-d} = v_1, \text{ and } v_2 - v_1 = v, \text{ as before.}$$

The new quantity, $Q' = D\,v_1 = \frac{Q\,D}{D-d}$.

The formula for the hydraulic jump is,

$$\frac{D^2 - d^2}{2} = \frac{Q'}{g}(v_2 - v_1)$$

and as D and d are unchanged, we may substitute for the foregoing values of v_2, v_1, and Q', their equivalents in terms of Q and v, as follows:

$$\frac{D^2 - d^2}{2} = \frac{Q\,D\,v}{g\,(D-d)} = \frac{D\,d\,v^2}{(D-d)\,g},$$

thus disclosing the identity of the two formulas and justifying, to some extent, the reasoning outlined in the premises.

To complete the analogy, it may come to mind that, as water cannot "jump" unless it has a velocity greater than $\sqrt{gd}$, it would be well to demonstrate that the sum of the velocities, v and v_3, is always greater than $\sqrt{gd}$.

Note, from Equation (1), that,

$$\frac{v^2}{g} = \frac{(D+d)\,(D-d)^2}{2\,D\,d}$$

and we are to show that

$$v + v_3 \text{ or } \frac{v\,D}{D-d} > \sqrt{gd},$$

that is, that

$$\frac{v^2}{g} > \frac{d\,(D-d)^2}{D^2},$$

or, eliminating $\frac{v^2}{g}$, that

$$\frac{D+d}{2\,d} > \frac{d}{D},$$

which is obvious so long as $D > d$.

The surge, S, above the level of the quiet water previous to its acceleration into the canal entrance is evidently equal to $D - d - \frac{v^2}{2\,g}$, and it may be shown by calculus methods that the maximum possible

value of S is equal to $0.714d$, which occurs for a critical velocity of $v = 7.448\sqrt{d}$.

Modifications Involving Friction.—It now seems clear that Equation (1) represents the relation between the depths on each side of the backward rolling wave when friction is neglected. Without attempting to go further into the subject at this time, it may be stated, nevertheless, that the surge probably cannot exceed the value of S derived from this equation, when friction is taken into account. On the other hand, there would seem to be little danger of extravagance if the height of the canal and forebay walls was regulated by the foregoing considerations.

DISCUSSION

Mr. Kennison.

KARL R. KENNISON,* ASSOC. M. AM. SOC. C. E. (by letter).—The author's determination of the height of the surge in an open canal, following a sudden interruption of the flow, appears to be based on sound theory. It is particularly interesting to the writer on account of the intimate relationship between this surge and the hydraulic jump, the discussions of which are published with the writer's paper "The Hydraulic Jump, In Open-Channel Flow at High Velocity".† The author's treatment of the hydraulic theories involved is complete, and requires little to be said in addition. The same conclusions, however, may be reached in a different way, at the same time bringing out some interesting characteristics of the hydraulic jump which were suggested by reading this paper; also, a formula for the canal surge is submitted herewith which is simpler than that deduced by the author.

There are points of difference between the ordinary hydraulic jump or standing wave and the author's receding wave which at first seem inconsistent, but which are really in agreement. It has already been shown† that in an open channel, carrying a certain quantity of water under a certain head, there are only two surface levels at which the water can flow steadily. If the velocity is less than $\sqrt{g \times \text{depth}}$, it is flowing at the upper alternative stage, and, if a dam of the proper height is interposed, it will drop to the lower alternative stage. If the velocity is greater than $\sqrt{g \times \text{depth}}$, it is already at the lower stage, and may jump to the upper stage by meeting either an obstruction, which, if of the right height and smoothness, may avoid all but incidental eddy losses, or a change in channel conditions sufficient to cause the normal jump with its eddy losses, as ordinarily observed. The question may arise: how can the level in a low-velocity canal, which is already at the upper alternative stage, jump any higher on the sudden closing of a gate, even higher than the level of quiet water before its acceleration into the canal entrance, as the author states?

The explanation is that the conclusions previously drawn, with reference to the hydraulic jump, assumed that the jump was in every case stationary, not moving up or down stream. Now, if this standing wave travels along the channel, we may, since velocity is only relative, correct all the velocities by an amount equal to the velocity of the wave, and then the conclusions regarding the hydraulic jump apply correctly to all such moving waves. For example, a suddenly interrupted canal flow, though flowing apparently at the upper low-velocity stage, is approaching the (receding) wave at so high a relative velocity that it is relatively at the lower

* Providence, R. I.

† *Transactions*, Am. Soc. C. E., Vol. LXXX, p. 338.

Mr. Kennison.

stage and capable of jumping higher. In fact, when we consider the standing wave or jump as movable along the stream, instead of stationary, there are, instead of two, an indefinite number of possible water levels. It can even be shown that absolutely still water in an open channel can theoretically be made to drop to any lower level or to rise to any higher level by the passage of a standing wave.

This relationship between the hydraulic jump and the surge in an open canal is already clear to one who has followed the author's admirable mathematical analysis. At the risk of some uninteresting repetition, an attempt is made to say the same thing in a different way, and also to show graphically some peculiarities of the hydraulic jump and its relation to the canal surge. The writer has found that elementary diagrams like these are often helpful in getting a clear idea of the subject.

In Fig. 1, two smooth obstructions or dams are assumed to be kept a uniform distance apart and moved along the bottom of a rectangular flume containing still water, with the result that the water level drops, as shown, and rises again to still water, neglecting, of course, friction and incidental eddies. The dimensions are chosen so that direct comparison may be made, if desired, with Figs. 3 to 8 in the writer's paper, "The Hydraulic Jump, In Open-Channel Flow at High Velocity". Higher dams moved at lower velocity would cause a drop lower than shown, and lower dams at higher velocity a drop not as low. The same height of dams, moved much more slowly than shown, would cause only a local depression over each dam. They could not be moved faster without raising the level of the still water ahead, until their velocity is increased to that shown in Fig. 2. Then the water would rise theoretically as shown and drop again to still water, neglecting friction and incidental eddies, which, of course, would actually be considerable at this velocity. In Fig. 3, one of the dams is replaced by a gate; and, in Fig. 4, the other dam is removed, resulting in a case exactly similar to the suddenly interrupted canal flow described by the author.

All velocity being relative, the absolute velocities in Fig. 4 are also expressed relative to the velocity of the standing wave, illustrating the normal hydraulic jump, and also relative to the gate, illustrating the canal surge. These four figures are not necessary to show this relation, but they may prove interesting in a study of the hydraulic principles involved.

It is apparent, therefore, that to find a general expression (neglecting friction) for the resulting depth, D, in a channel of rectangular cross-section in which the water, flowing with depth, d, and velocity, v, is suddenly checked, it is merely necessary to take Professor Unwin's formula for the hydraulic jump,* which is in excellent agreement with

* *Transactions*, Am. Soc. C. E., Vol. LXXX, p. 410.

Mr. Kennison.

experiment and is apparently based on sound theory, and substitutes for the velocity before the jump its value in terms of the difference in velocities before and after the jump. The result checks exactly with the equation deduced independently by the author. Since this

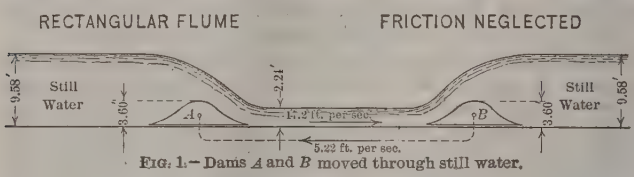

FIG. 1.—Dams A and B moved through still water.

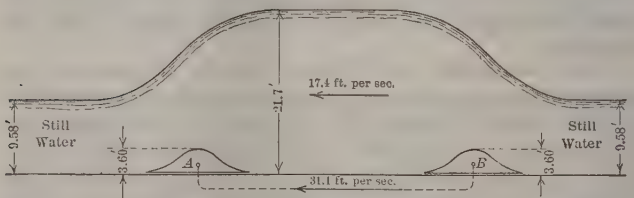

FIG. 2.—Same as Fig. 1, except that A and B are moved about six times as fast.

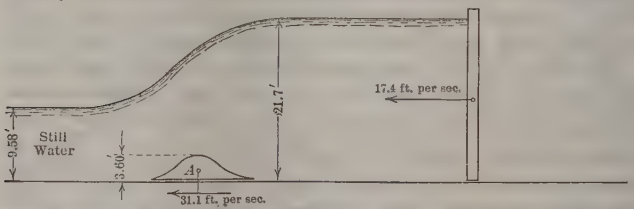

FIG. 3.—Same as Fig. 2, except that B is replaced by gate moving about half as fast.

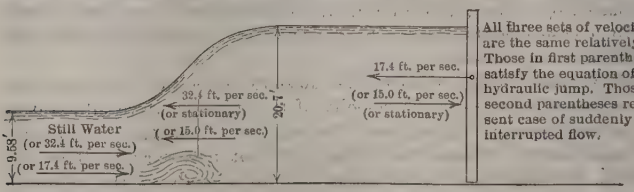
All three sets of velocities are the same relatively. Those in first parentheses satisfy the equation of the hydraulic jump. Those in second parentheses represent case of suddenly interrupted flow.
17.4 ft. per sec.
(or 15.0 ft. per sec.)
(or stationary)
32.4 ft. per sec.
(or stationary)
20.1'
Still Water
(or 32.4 ft. per sec.)
(or 15.0 ft. per sec.)
(or 17.4 ft. per sec.)
9.58'

FIG. 4.—Same as Fig. 3, except that A is removed so that jump, instead of occuring without loss of head, contains the normal jump losses, the surface is not lifted so high, and the standing wave travels faster.

is a cubic equation, it cannot be solved easily, except by trial. The following simple equations will probably be found more convenient. They are not mathematical equivalents of the Unwin quadratic equation for the hydraulic jump and the author's cubic equation for the

canal surge, but, as shown by Table 1, for all reasonable uses, the error is well within the precision attainable in hydraulic computations of this nature. d is the depth, in feet, and v is the velocity, in feet per second, in a rectangular (frictionless) flume. D is the depth after the jump. Mr. Kennison.

For the hydraulic jump or standing wave:

$$D = \frac{v\sqrt{d}}{4} - 0.45\,d \quad \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

For the receding wave caused by sudden interruption of flow:

$$D = \frac{v\sqrt{d}}{5} + 0.99\,d \quad \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

TABLE 1.—Approximate Values of $D \div d$.

By the Unwin or Johnson Formulas.	By Equation (1).	By Equation (2).
1.20	1.18	1.21
1.40	1.39	1.41
1.60	1.60	1.60
2.00	2.00	1.97
4.00	4.03	.....
6.00	6.05	.....
10.00	10.07	.....
20.00	20.10	.....

Irving P. Church,* Assoc. Am. Soc. C. E. (by letter).—As regards the very interesting problem involved in this paper, on the surge produced in a nearly level, open canal of rectangular section, in which water is initially flowing with uniform velocity, when a vertical gate is suddenly dropped and completely closes the channel, the writer recalls no prior treatment in English, except as found on the last page of an article by Mr. Ford Kurtz.† In his treatment, however, in applying the method involving "change of momentum" Mr. Kurtz inadvertently used the mass of the flow per second of the water approaching the advancing surge and as yet unaffected by it, instead of the mass suffering impact per second, in forming his expression for the rate of change of momentum; that is (in his notation), he wrote $\frac{\gamma\, b\, h_1\, v_1}{g}$, instead of $\frac{\gamma\, b\, h_1}{g} \times \frac{l_1}{t}$. Had he used the latter expression, he would have arrived at a result identical with Mr. Johnson's Equation (1). Mr. Church.

Turning to French sources: Flamant‡ gives the following demonstration (here modified for a channel of rectangular section and

* Ithaca, N. Y.

† "Application of Newton's Second Law of Motion to Certain Hydraulic Problems". *The Cornell Civil Engineer*, January, 1916. Published at Cornell University, Ithaca, N. Y.

‡ "Hydraulique", p. 415, Second ed., Paris, 1900.

Mr. Church.

constant width, b, and with some of the notation of the present paper). See Fig. 5.

Let AB be a rigid vertical plate, or gate, entirely closing the end of a straight horizontal channel of rectangular section and constant width, b, which contains water at rest, extending indefinitely to the right, and of depth, d. The plate is at right angles to the sides of the channel. Let the plate now assume instantaneously a velocity of U feet per second toward the right, this velocity being then maintained at constant value. The water near the plate acquires (in small installments) the same velocity, U, and is heaped up in front of it with a flat top and constant depth, D, the front edge, $C''C$, of this wave (wave-front) moving toward the right with some constant velocity, V. Let $D - d$, or the height of the wave, be denoted by h. If γ is the weight of a cubic unit of water, the total horizontal pressure (above atmospheric) between the plate and the raised water is $\frac{\gamma\, b\, D^2}{2}$; and, similarly, that between the vertical face, CE, of the water just under the wave-front and the (as yet) stationary water on its right is $\frac{\gamma\, b\, d^2}{2}$.

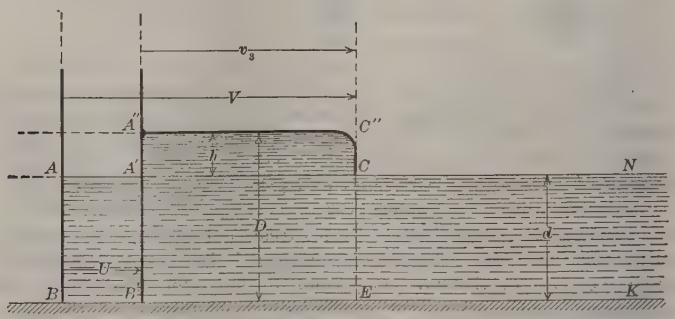

Fig. 5.

Let us now trace the motion for the first second of time. At the end of this second the plate is at $A'B'$, having moved a distance $\overline{AA'} = U$, the wave-front is at C, at a distance $\overline{AC} = V$, from AB, and the parallelopiped of water, $A''C''CEB'A'A''$ has a velocity, U (the water on the right being still at rest); while at the beginning of this second, the plate being at AB, this same mass of water formed the parallelopiped, $ACEBA$, and had a velocity of zero. The change of momentum brought about in this mass during the first second, therefore, is $\frac{\gamma\, V\, b\, d}{g}\,(U - 0)$, which is also the rate of change of momentum,

since the time concerned is a unit. Hence, equating the sum of the horizontal components of the external forces to the rate of change of momentum in a horizontal direction (neglecting friction on the bed), we have Mr. Church

$$\frac{\gamma\, b\, D^2}{2} - \frac{\gamma\, b\, d^2}{2} = \frac{\gamma\, V\, b\, d}{g}\, U \qquad \ldots\ldots(2)$$

Water being incompressible, we have also: volume, $AA'B'B$ = that of the horizontal "lamina", $A''C''CA'A''$; whence

$$Ud = (V - U)\,(D - d) \qquad \ldots\ldots(3)$$

or,

$$UD = V(D - d) \qquad \ldots\ldots(3a)$$

that is, replacing $D - d$ by Flamant's symbol, h (height of wave), we may also write

$$Ud = (V - U)h \qquad \ldots\ldots(3b)$$

and

$$UD = Vh \qquad \ldots\ldots(3c)$$

Eliminating U by means of Equations (2) and (3b), and writing h for $D - d$, we may solve for V, obtaining

$$V = \sqrt{g\left[d + \frac{3}{2}h + \frac{1}{2} \times \frac{h^2}{d}\right]} \qquad \ldots\ldots(4)$$

or, approximately, since the last term within the brackets is generally quite small compared with those preceding,

$$V = \sqrt{g\left[d + \frac{3}{2}h\right]} \qquad \ldots\ldots(4a)$$

or again, expanding $\left[1 + \frac{3}{2} \times \frac{h}{d}\right]^{\frac{1}{2}}$, and retaining only the first two terms of the converging series (that is, neglecting $\left(\frac{h}{d}\right)^2$ and higher powers as compared with the first two terms), we derive another approximate relation, *viz.*:

$$V = \left[1 + \frac{3}{4} \times \frac{h}{d}\right] \sqrt{g\, d} \qquad \ldots\ldots(4a)'$$

Flamant does not attempt to solve for the wave height, h, in terms of U and d, but is principally interested in the value of V, the velocity of wave propagation; but, if we substitute in Equation (2) the value of V derived from Equation (3a), there is obtained

$$\frac{D^2 - d^2}{2} = \frac{D\, d}{D - d} \times \frac{U^2}{g} \qquad \ldots\ldots(5)$$

a cubic in D, if D (and finally h, $= D - d$) is sought; U and d being given.

Mr. Church.

Leaving Flamant's demonstration, it is noted that, relatively to the moving plate, the still water on the right of CE has a velocity toward the plate (call this velocity v) equal to U; and, similarly, that the velocity (call it v_3) of the wave-front, $C''CE$, relatively to the moving plate, is $A\,C$, that is, $V - U$, away from the plate. It follows, therefore, that in case the water in the open channel, with depth, d, has originally a velocity of v feet per second toward the left and the plate is suddenly dropped, so as to block completely the flow toward the left, and remains fixed in that position, the values of the wave-front velocity, v_3, and of D (or of wave height, h), become determinable by simply substituting v for U, and $v_3 + v$ for V, in the preceding equations. v_3, therefore, is the velocity of the wave-front, or surge, and D is the depth of the (now motionless) water between the plate and the wave front. This conception, by which the solution of the surge problem may be based on that of the plate advancing against still water, is introduced by the French engineer, Bazin, in a report (to be referred to later) on the experimental investigation of both cases.

In this way, then, we obtain for the problem of the surge wave dealt with by Mr. Johnson,

$$\frac{D^2 - d^2}{2} = \frac{D\,d}{D - d} \times \frac{v^2}{g} \qquad (6)$$

(which checks the author's Equation (1)), and

$$v_3 = \sqrt{g\left[d + \frac{3}{2}\,h + \frac{1}{2} \times \frac{h^2}{d}\right]} - v \qquad (7)$$

In these equations, and also in all subsequent formulas, the author's notation is used, with the addition of $h = D - d$. We may also set down for this case the approximate relations (see Equations (4*a*) and (4*a*)'):

$$v_3 = \sqrt{g\left[d + \frac{3}{2}\,h\right]} - v \qquad (8)$$

and

$$v_3 = \left[1 + \frac{3}{4} \times \frac{h}{d}\right]\sqrt{g\,d} - v \qquad (9)$$

As regards solving separately for the height of surge, h, or $D - d$, we may write $v_3 + v$ for V, and v for U, in Flamant's Equation (2), combine with Equation (9), and then solve the resulting quadratic; whence, if $v\sqrt{\frac{d}{g}}$ be denoted by k, there results

$$h = \frac{1}{4}\left[\sqrt{16\,d^2 + 8\,k\,d + 9\,k^2} + 3\,k - 4\,d\right] \qquad (10)$$

as a fair approximation when h is small compared with d.

Mr. Church.

We have, also, from Equation (3*b*)

$$vd = v_3 h \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (10a)$$

In his report* to the French Academy of Sciences on an experimental investigation of the propagation of waves in open channels, Bazin gives the data and results of Bidone (1824) in the same field as well as Darcy's (1856). Bidone's experiments were performed on a very small scale, his channel being only about 2 ft. wide and about 40 ft. long in which to create and observe the wave phenomena; whereas Darcy and Bazin made use, not only of an experimental channel about 6.5 ft. wide and more than 1 000 ft. long, with depths of water of 2 ft. and less, but also of a straight reach of a navigation canal, some 3 000 ft. long and 30 ft. wide, with depths as great as 3 ft., as well as of a smaller basin 20 ft. wide.

Such being the fairly large scale on which the Darcy experiments were made, it has always seemed remarkable to the writer that Bidone's results should be quoted so frequently in American books on hydraulics, with little or no mention of Darcy's (in this field of wave motion, standing waves, etc.).

According to this report of Bazin's, Bidone derived the following relations from his experiments with surges going up stream and caused by the abrupt closing of a transverse gate in moving water, channel rectangular, *viz.*:

$$h = \frac{2d + h}{h} \times \frac{v^2}{2g} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (11)$$

which may be written $h = \frac{v^2}{2g}\left[\frac{1}{2} + \sqrt{\frac{4gd}{v^2} + \frac{1}{4}}\right] \ldots\ldots\ldots\ldots\ldots (12)$

and $$v_3 = \frac{vd}{h} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (13)$$

As to the formulas based by Bazin on the Darcy experiments, let us first note that in 1844 J. Scott Russell presented, at the Fourteenth Meeting of the British Association for the Advancement of Science, an account of his interesting experiments on the "wave of translation" in still water. These were made on quite a small scale, in a channel of rectangular section, and led to the formula, $V = \sqrt{g(d + h)}$, for the velocity of the wave. (Compare with Flamant's Equation (4*a*).) This result was verified very satisfactorily by the Darcy experiments on the same phenomenon, these being made on a much larger scale. Bazin, therefore, adopted this formula, modified to suit the altered conditions, as a foundation for an expression for the velocity of wave propagation for the case now under discussion (sudden complete

* "Recherches Hydrauliques"; by Darcy and Bazin, Deuxième Partie, Paris, 1865.

Mr. Church. closing of a gate across a rectangular channel containing moving water). That is, he first writes

$$v_3 = \sqrt{g\,(d + h)} - v \qquad (14)$$

and then eliminates h by the aid of the relation,

$$h\,v_3 = v\,d \qquad (15)$$

see Equation (10a). The resulting cubic in v_3 can be factored, and yields one positive root, *viz.*:

$$v_3 = \sqrt{g\left(d + \frac{v^2}{4\,g}\right)} - \frac{v}{2} \qquad (16)$$

But, experiment showing a somewhat larger value for h in terms of v_3 than as given by Equation (15), he modifies the algebraic relations on that basis and finally obtains, as fairly justified by experiment,

$$v_3 = \sqrt{\frac{v^2}{4} + g\,d} - \frac{2}{5}\,v \qquad (17)$$

and, as an average,

$$h = \frac{1.2\,v\,d}{v_3} \qquad (18)$$

As it may be of interest to compare the results obtained from these various formulas, let us take the data: $v = 4$ ft. per sec., and $d = 4$ ft., to find both v_3 and h (that is, $D - d$). The results are shown in Table 2.

TABLE 2.

Equation from which derived.	h, in feet.	v_3, in feet per second.
Equation (6), Johnson	1.518	.....
(7) "	.....	10.53
(8)	.....	10.22
(9)	.....	10.56
(10)	1.52	.....
(12), Bidone	1.544	.....
(13) "	.....	10.39
(17), Bazin	.....	9.92
(18) "	1.93	.....

Bazin calls attention to an important fact which escaped the attention of Bidone: that the height of wave at the wave-front itself is somewhat greater than that of the portion of water behind. Bazin's formula seems to provide for this.

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Paper No. 1382

THE PROPERTIES OF BALSA WOOD
(*Ochroma Lagopus*)*

By R. C. Carpenter, M. Am. Soc. C. E.

With Discussion by Messrs. A. P. Lundin and Leonard M. Cox.

Synopsis.

Balsa wood grows extensively in the Central American and northern South American States as a second-growth tree. It is the lightest wood known, so far as any evidence attainable is concerned. This paper shows the microscopical structure and also gives various tests of its transverse and compressive strength. The material is composed of very thin-walled cells, which are barrel-shaped, interlace with each other, and are almost devoid of woody fiber. These cells are filled with air, making a natural structure well adapted to prevent the transmission of heat, because of the particles of air imprisoned in the material without interconnecting fibers. Various tests of the insulating properties for resisting the flow of heat are given.

Balsa wood has been used quite extensively in the past as a buoyancy product for life preservers and in connection with the fenders of life-boats and rafts. Its life is short, under ordinary conditions, unless treated with antiseptic or preservative material. There is promise that it will have a field of usefulness, in the lines referred

* Presented at the meeting of June 7th, 1916.

to, which must make it of considerable interest to the engineer who requires either insulating material or buoyancy products.

Very little information is available respecting the wood of the balsa tree, which has recently been applied to several practical uses, and, as it possesses properties which make it valuable for many engineering purposes, the writer has thought it of sufficient interest to warrant the publication of the information which he has obtained regarding it. His interest in the wood was excited as the result of investigations as to its properties undertaken more than a year ago, and recently also by a visit to the Isthmus of Panama, where it grows extensively. The wood is remarkable: first, as to its lightness; second, as to its microscopical structure; third, as to the absence of woody fiber; fourth, as to its elastic character, in the sense of recovery from transverse deformation; and fifth, for its insulation qualities for heat. It is the lightest wood commercially useful, so far as the writer has been able to ascertain, and it has considerable structural strength, which makes it suitable for a fairly extensive use.

The Weight of Balsa Wood.

Balsa wood, when thoroughly dried, has a specific gravity of 0.11. For reference, Table 1* shows the relative weights of various woods. Until recently, Missouri cork wood, weighing 18.1 lb. per cu. ft., was supposed to be the lightest, but recent investigations indicate that balsa wood is much lighter, having a weight of 7.3 lb. per cu. ft. The ordinary commercial balsa wood is seldom perfectly dry, and, because of the moisture content, its weight, as appears from a number of investigations made by the writer, will usually be between 8 and 13 lb. per cu. ft. As will be seen from Table 1, however, it is much lighter than cork.

Cellular Structure of Balsa Wood.

The cellular structure of balsa wood, as exhibited under a microscope, differs from that of any other wood known to the writer. All engineers know that wood is made up of a series of interlacing cellular bodies of microscopic size which, when joined together, form fibers which extend both radially and longitudinally. These cellular fibers

* From the *Bulletin* of the Missouri Botanical Garden, August, 1915.

are interlaced, and, by their form and arrangement, give the wood its strength and physical properties. In ordinary woods the thickness of the walls of the cells is generally a considerable proportion of the diameter. The cells which are parallel to the axis of the tree are made up principally of woody fiber; those which extend in a radial direction usually have a cellulose structure with little woody fiber, and are defined as "medullary rays," or pith cells, because of their position and composition. The microscopic structure of all the woods involves, in addition, the existence of ducts or vessels scattered through the wood in a longitudinal direction; these serve as a circulatory system for the transmission of liquids and gases during the growth of the tree.

TABLE 1.—Weights of Woods.

Common name.	Scientific name.	Weight, in pounds per cubic foot.
Balsa	*Ochroma lagopus*	7.3
Cork	(Bark from cork oak, *Quercus suber*)	13.7
Missouri cork wood	*Leitneria floridana*	18.1
White pine	*Pinus strobus*	23.7
Catalpa	*Catalpa speciosa*	26.2
Cypress	*Taxodium distichum*	28.0
Douglas fir	*Pseudotsuga mucronata*	32.4
Sycamore	*Platanus occidentalis*	35.5
Red oak	*Quercus rubra*	40.5
Maple	*Acer saccharum*	43.0
Long-leaf pine	*Pinus palustris*	43.6
Mahogany	*Swietenia mahogoni*	45.0
Locust	*Robinia pseudo acacia*	45.5
White oak	*Quercus alba*	46.8
Hickory	*Carya alba*	54.2
Live oak	*Quercus virginiana*	60.5
Ironbark	*Eucalyptus leucoxylon*	70.5
Lignum-vitæ	*Guaiacum sanctum*	71.0
Ebony	*Diospyrus*	73.6
Black ironwood	*Krugiodendron ferreum*	81.0

Figs. 1 to 6 are reproductions from micro-photographs of balsa (*Ochroma lagopus*). Figs. 1 and 2 show cross-sections, Figs. 3 and 4 radial sections, and Figs. 5 and 6 tangential or longitudinal sections. In these illustrations, the ducts or vessels are denoted by (*a*), the medullary rays by (*b*), and the barrel-shaped cells which constitute the longitudinal fiber, by (*c*).

To W. W. Rowlee, Professor of Botany in Cornell University, who assisted in the investigations, the writer is indebted for the micro-photographs and also for the following botanical description:

"*Gross Characteristics.*—In general appearance, balsa wood resembles basswood. As shown by the accompanying micro-photographs,

Figs. 1 to 6, its medullary or pith rays (*b*) are uniformly spaced, and are quite prominent in both the radial and cross-sections. In the radial sections, Figs. 3 and 4, they appear much as in maple or sycamore, as well as basswood, but lack the hardness and susceptibility to polish possessed by these woods. The ducts, pores, or vessels, shown at (*a*), are large and remote from each other, and occur singly, or in groups, in the strands between the pith rays (*b*).

"The lightness of the wood is one of its most striking features. This is due to the thinness of the walls of the elements. There is rather indistinct evidence of annual rings in the cross-section. In the specimens studied, the regular concentric rings, so characteristic of trees of temperate regions, do not show.

"*Minute Structure.*—The pith, or medullary ray cells (*b*) have normal position and form, but the cells are not elongated radially to so great an extent as is usually found in woods. The ducts (*a*) are large, with rather thin, pitted walls. Woody fibers of the ordinary sort seem to be absent in this wood, their place being taken by a cellulose tissue (*c*) very much like the thin-walled tissue of the pith and cortex of ordinary trees.

"The cells (*c*) making up this tissue are barrel-shaped, whereas woody fibers are taper-pointed and relatively much longer. The most remarkable thing about them, however, is their exceedingly thin, unlignified walls. A section of the tissue in question, examined under a microscope, would not be taken for wood, but rather thin-walled cells or 'parmchyma' from the pith or cortex of a stem. The only lignified part is the wall of the duct, and that is relatively weak.

"*Conclusions.*—

"1.—The gross structure of balsa wood is in appearance like basswood, poplar, or willow.

"2.—Its weight shows that it is fundamentally different from these.

"3.—Its minute anatomy is radically different from any wood known to the writer.

"4.—What correspond to the woody fibers, shown at (*c*), are not lignified. They are very thin-walled and soft.

"5.—The ducts or pores, shown at (*a*), are weakly lignified, and are pitted. They, however, constitute a very small proportion of the wood.

"6.—The pith rays, shown at (*b*), are also thin-walled and not lignified."

Microscopical Structure of White Pine (*Pinus Strobus*).—For the purpose of illustrating the interlacing fiber construction of ordinary woods, the writer presents micro-photographs of white pine, Figs. 7, 8, and 9, magnified about 100 diameters. Sections of the longitudinal cells are shown in Fig. 9 at *c c*; these have pointed ends, and are

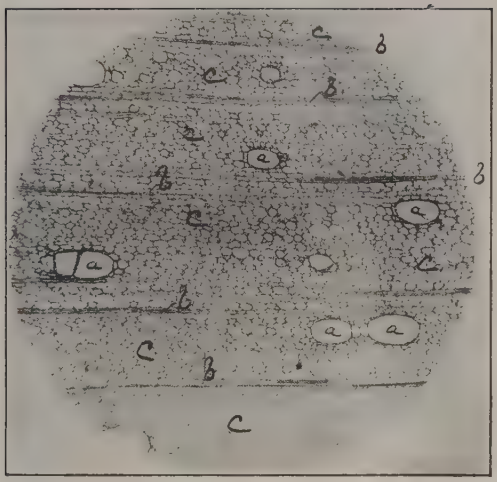


FIG. 1.—(*Ochroma lagopus*) CROSS-SECTION, BALSA, ABOUT 78 DIAMETERS, SHOWING (*a*) LARGE VESSELS, (*b*) MEDULLARY RAYS, (*c*) CROSS-SECTION OF LONGITUDINAL CELLS.

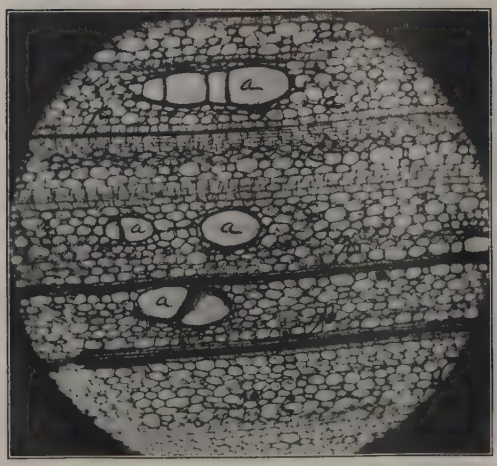


FIG. 2.—CROSS-SECTION, BALSA (*Ochroma lagopus*), SHOWING (*a*) LARGE VESSELS, (*b*) MEDULLARY RAYS, (*c*) CELLS CONSTITUTING LONGITUDINAL FIBERS.

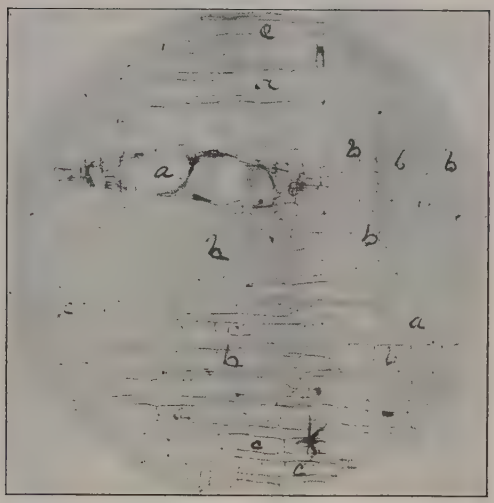


FIG. 3.—RADIAL SECTION, BALSA, ABOUT 70 DIAMETERS, SHOWING (*a*) LARGE VESSELS OR DUCTS, (*b*) MEDULLARY RAYS, (*c*) LONGITUDINAL CELL STRUCTURE.

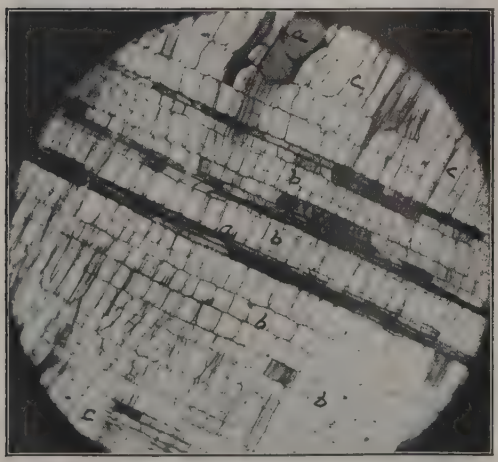


FIG. 4.—RADIAL SECTION, BALSA, ABOUT 130 DIAMETERS, SHOWING (*a*) VESSELS, (*b*) MEDULLARY RAYS, (*c*) CELLS CONSTITUTING LONGITUDINAL FIBERS.

FIG. 5.—TANGENTIAL SECTION, BALSA, ABOUT 70 DIAMETERS, SHOWING (*a*) LARGE VESSELS OR DUCTS, (*b*) MEDULLARY RAYS IN CROSS-SECTION, (*c*) CELLS CONSTITUTING BARREL-SHAPED LONGITUDINAL FIBERS.

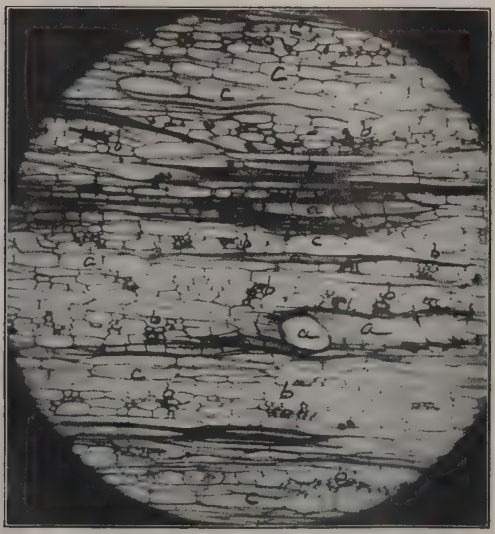


FIG. 6.—TANGENTIAL SECTION, BALSA, ABOUT 110 DIAMETERS, SHOWING (*a*) VESSELS, (*b*) MEDULLARY RAYS, (*c*) CELLS CONSTITUTING LONGITUDINAL FIBERS. NOTE BARREL FORM AND THIN WALL OF LONGITUDINAL CELLS.

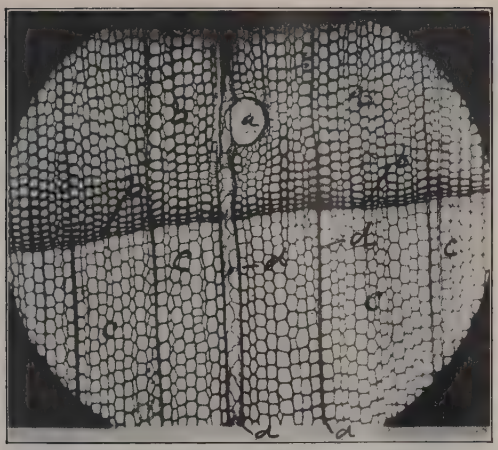


FIG. 7.—*Pinus strobus* (WHITE PINE), ABOUT 100 DIAMETERS. CROSS-SECTION AT ANNUAL RING, *b* (OLD CELLS, UPPER PART WITH THICK WALLS, NEW CELLS BELOW WITH THIN WALLS). (*a*) VESSELS OR DUCTS, (*b*) LONGITUDINAL CELLS (OLD), (*c*) LONGITUDINAL CELLS (NEW), (*d*) MEDULLARY RAYS.

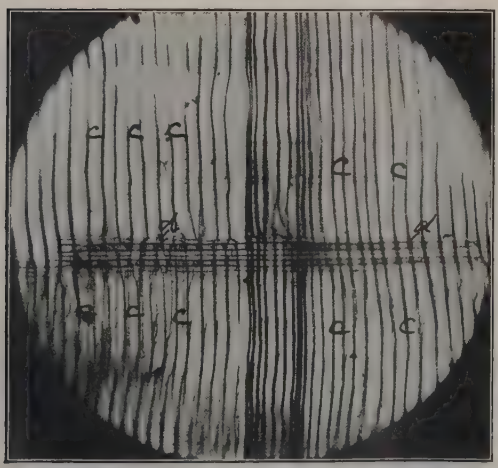


FIG. 8.—*Pinus strobus* (WHITE PINE), ABOUT 100 DIAMETERS, RADIAL SECTION AT ANNUAL RING SHOWING (*d*) MEDULLARY RAYS, AND (*c*) PITTED TRACHEIDS OR LONGITUDINAL CELLS.

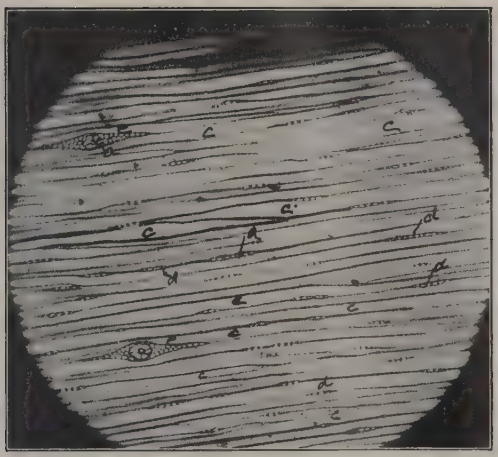


FIG. 9.—*Pinus strobus* (WHITE PINE), ABOUT 100 DIAMETERS. LONGITUDINAL TANGENTIAL SECTION SHOWING (*d*) COMPOUND MEDULLARY RAYS, AND (*c*) PITTED TRACHEIDS OR LONGITUDINAL POINTED CELLS. THE LONGITUDINAL CELLS EXTEND BEYOND THE LIMITS OF THE FIGURE.

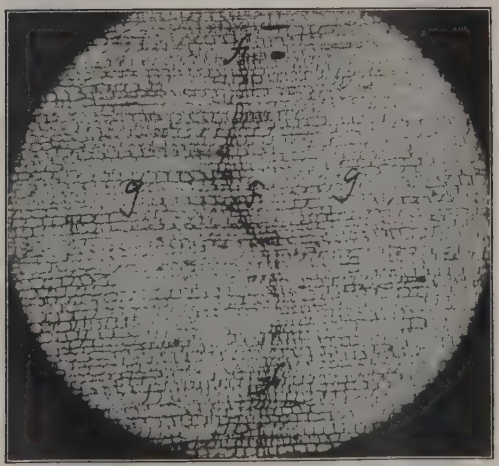


FIG. 10.—*Quercus suber* (CORK OAK) BARK, ABOUT 100 DIAMETERS. CROSS-SECTION AT ANNUAL RING SHOWING (*f*) ANNUAL RING, (*g*) CROSS-SECTION OF CELLS.

FIG. 11.—*Quercus suber* (CORK OAK) BARK, ABOUT 100 DIAMETERS, RADIAL SECTION AT ANNUAL RING SHOWING (*f*) ANNUAL RING, (*g*) RADIAL SECTION OF CELLS.

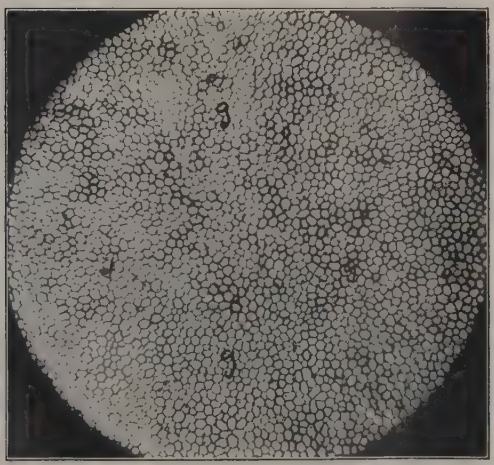


FIG. 12.—*Quercus suber* (CORK OAK) BARK, ABOUT 100 DIAMETERS, TANGENTIAL SECTION SHOWING (*g*) RADIAL CROSS-SECTION OF CELLS.

many times longer than the field of the microscope. They have walls of woody fiber which are much thinner in pine than in hard woods like oak, but are thicker than in balsa. The cells constituting the medullary rays are shown at *d* in Figs. 7, 8, and 9; they are devoid of woody fiber, and are much shorter than the longitudinal cells, *c*, but they are longer than any of the cells in balsa wood. Pine has a very different cellular structure from balsa, as shown by the micro-photographs. In pine the longitudinal cell walls are composed principally of woody fiber, and the medullary ray cells of cellulose. The cells of the older wood, near the annual ring, have very thick walls in pine, as appears from Fig. 7. Evidently, the cellular structure of white pine is characteristic of a heavier wood, of less insulating capacity than balsa, and in that respect predicates the difference in physical tests and in insulating properties.

The Microscopical Structure of Cork Bark.—As illustrating a wood product, already in extensive use for heat insulating purposes, which has a cellular structure without interlacing fibers, attention is called to the micro-photographs of bark from the cork oak, *Quercus suber*, Figs. 10, 11, and 12. These interesting illustrations show that cork bark is composed of a series of thin-walled cells of small dimensions, without the interlacing fiber characteristics, as in the woods considered. The balsa wood, with respect to this feature, may be considered as intermediate between pine and cork bark. The value of cork and balsa for insulating purposes is evidently due to the close union of numerous closed cells filled with air. The deficiency of cork in structural strength is due to the absence of interlacing fibers which characterize balsa and other woods. It represents the extreme of cellular structure. The cork bark contains, interspersed between the cellular structure, a resinous deposit which is utilized, by aid of pressure and heat, in forming cork board, which is usually put on the market in blocks, 2 or 3 in. thick and from 2 to 6 ft. long, convenient for use as a lining in cold-storage or other structures. Cork is soft and readily compressible, and represents extreme characteristics possessed by balsa wood in a less degree.

The Strength of Balsa Wood.

Table 2 shows tests made under the writer's direction at Sibley College, Ithaca, N. Y., and also as reported by Professor Walter S. Leland, formerly of the Massachusetts Institute of Technology.

TABLE 2.—Transverse Tests of Balsa Wood.

No.	Dimensions, in inches.	Modulus of rupture.	Deflection, in inches.	Quality.	Where made.
A	1¾ by 2½ by 20	2 880		Medium	Cornell.
B	1¾ " 2½ " 20	3 290		Clear	"
C	1¾ " ½ " 20		0.847	Clear	"
D	1¾ " ½ " 20		1.123	Clear	"
1	5 by 5 by 96	3 500	2	Clear	M. I. T.
2	1¾ " 4½ " 96	3 600	1 5/16	Poor	"
3	1¾ " 4½ " 96	2 900	1¼	Very poor	"
4	3 1/16 " 4⅝ " 96	3 300	2 9/32	Clear	"
5	1⅞ " 5⅛ " 96	3 207		Clear	"

Weight of Specimen *A*.....13.19 lb. per cu. ft.
" " " *B*.....10.05 " " " "
Average weight of Nos. 1–5.....13.2 " " " "

*Crushing and Compression Tests.**—Three specimens, each 1¾ by 2½ by 4 in., with a cross-section of 4.375 sq. in., gave an average of 2 488 lb. per sq. in. Another test of three specimens gave an average of 2 225 lb. per sq. in.

Three specimens, 1 by 1 by 3 in., crushed with loads of 2 210, 2 380, and 2 530 lb. per sq. in., respectively.

Two compression tests, of specimens 1 by 1 by 16 in., showed maximum loads of 1 860 and 1 980 lb., and net compression of 0.69 and 0.55 in., respectively.

One specimen, 5⅞ by 5⅞ by 23 25/32 in., crushed under a load of 40 900 lb., equivalent to 2 500 lb. per sq. in.; and one slightly smaller gave substantially the same strength per unit of section as reported by Professor Leland in Table 2.

Professor Leland states:

"The crushing strength seems to be very satisfactory for such wood—about one-half the strength of white pine or spruce.

"These tests show the modulus of rupture to be approximately one-half that of good spruce, and their uniformity clearly shows that the material may be relied on both for direct compression and transverse loads.

"It is very elastic material, and when the load was almost at the breaking point, the load on three of the beams was removed and the beams resumed their original shape.

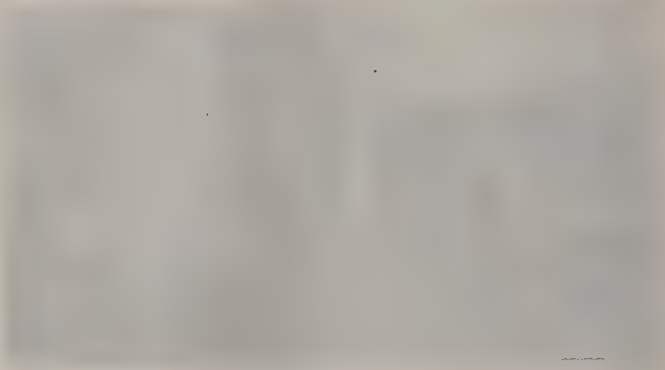
"It is exceedingly interesting to note that it is practically impossible to split the wood by driving nails through it."

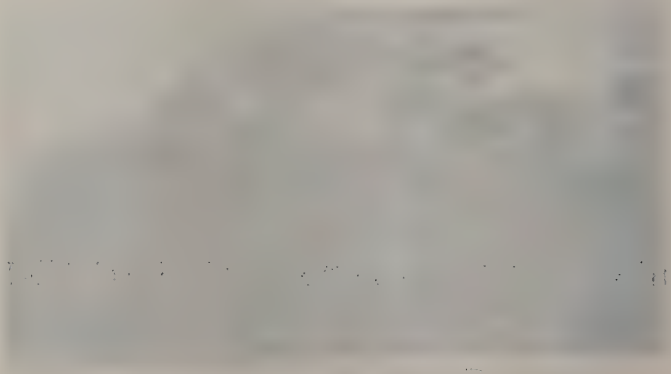
Fig. 13 shows a balsa wood plank supported on horses and carrying two men, the plank being 5½ in. wide, 1¾ in. thick, and 10 ft. 8 in.

* Except where otherwise noted, these tests were made at Cornell University.

FIG. 13.—DEMONSTRATION OF THE ELASTIC DEFORMATION OF BALSA WOOD.

FIG. 14.—APPEARANCE OF YOUNG BALSA TREES, COSTA RICA.





between supports. The weights carried were 187 and 200 lb., respectively. The maximum deflection at the center was about 10 in.

The Habitat.

In a recent trip to the Isthmus of Panama the writer found balsa trees growing commonly in all the cleared spaces which were not under cultivation in the Canal Zone. Most of these trees were of small diameter, and evidently quite young, and in every instance they were found in the newly started jungle which has recently been allowed to grow over a goodly part of the Canal Zone since the canal has been completed. In some cases these trees were growing vigorously in the masses of material sliding into the canal. The tree is characterized by a large leaf, from 14 to 30 in. in greatest length, which has a form shown in Fig. 15, and by the peculiar seed pods which it bears when it reaches a larger size. Excellent evidence was obtained that the balsa tree grows very rapidly, and attains a diameter of from 12 to 14 in. at an age of 4 or 5 years. Rear-Admiral H. H. Rousseau, M. Am. Soc. C. E., now in charge of considerable construction work in the Canal Zone, told the writer that a balsa tree which was growing near his house at Culebra attained dimensions approximating 12 to 14 in. in diameter and from 40 to 60 ft. in height, in about 4 years. A considerable quantity of balsa of large size is to be found near the cleared plantations along the Chagres River, and in various other places near the Atlantic Coast and the banana plantations.

The result of an investigation of the forests of a number of tropical countries, by Mr. Herbert Paschke, undertaken for Capt. A. P. Lundin, indicates that balsa trees are found in considerable quantities in Honduras. Costa Rica, Colombia, and Jamaica, and there is abundant evidence that it grows vigorously in most of the tropical countries of South America. The report referred to indicates that the balsa or *ochroma* is entirely a second-growth wood, and is never found in the virgin forest, except as an isolated tree or two where clearing has occurred. The writer also learned, from his visit to the tropics, that forests composed of any one species do not exist in tropical countries, as they are found in the United States. Tropical trees always grow individually or by themselves, and very rarely in close proximity to other trees of the same species. This fact makes it necessary to spend considerable sums for transportation in gathering

any tropical timber, as great distances through the jungle have to be traversed in order to obtain the timber of a single tree. The general characteristics of the growing balsa tree are shown clearly in Fig. 14, reproduced from a photograph taken in Costa Rica.

Mr. Paschke states that balsa wood is known by different names in various American tropical countries, as follows:

In British Honduras.....“Moho”
In Guatemala.....“Lanillo” or “Moho”
In Spanish Honduras.....“Guano”
In Costa Rica.....“Balsa”
In Bocas del Toro.....“Moho”

Balsa wood, because of its lightness, has been known for a long time as a desirable material for floats, for supporting other material, and for rafts. Professor Gifford states that in the West Indies the natives use it for poles* “somewhat as the Chinese use the bamboo for shoulder poles, for all uses where a light, rather strong pole is needed.” So far as the writer could learn, balsa wood has no commercial value with people in the tropics who know about it, the general idea being that its lasting power is very slight, and that it warps and checks when exposed to the weather so as to be of little or no practical use.

Mr. W. F. Morgan, who recently made an investigation regarding the growth of balsa wood in Costa Rica, states that the natives are in the habit of cutting large balsa trees simply for the seed pods, which grow a woolly fiber suitable for pillows and mattresses. Fig. 16 shows a log of balsa wood, about 2 ft. in diameter, the tree having been cut down merely for the seed pods. Mr. Morgan also states that it has been the custom to use balsa wood in rafts of heavy timber for the purpose of securing buoyancy. At the end of a voyage, this wood is thrown away as having no commercial value.

It is thought that the first person to make any extended commercial use of balsa wood was Capt. A. P. Lundin, President of the Welin Marine Equipment Company, and formerly connected with the Pacific Mail Steamship Company. From his travels in tropical countries, Capt. Lundin knew of the extreme lightness of this wood, and its value as a buoyancy material in life preservers and life-boats was sug-

* von Schrenk.

FIG. 15.—THE LEAF OF THE BALSA TREE.

FIG. 16.—BALSA WOOD LOG. TREES CUT FOR SEED PODS.

gested by its properties. When he undertook to apply the wood practically, however, he found that it was of little value because it absorbed moisture in great quantities, and also because it soon rotted, and also warped and checked when worked. He then undertook the discovery of some means of treating the wood which would render it water-proof and also prevent it from changing its shape. After testing nearly every method that had been suggested, Col. Marr's method of treating woods, which had been recently patented, was finally successful. In this method the wood is treated in a bath, of which the principal ingredient is paraffin, by a process which coats the interior cells without entirely clogging up the porous system. The paraffin remains as a coating or varnish over the interior cell walls, preventing the absorption of moisture and the ill effects as to change of volume and decay which would otherwise take place; it also prevents the bad effects of dry rot, which follows the use of any surface treatment for preserving wood of the balsa type.

The Marr process tends to drive out all water and make the wood water-proof; it improves the quality of being readily worked with tools, without material increase of weight. The treated balsa wood has been used extensively by the Welin Marine Equipment Company in the manufacture of life preservers, fenders for life-boats, and for structures requiring insulation from heat, as in the refrigerating compartments of vessels, and in ice boxes.

Transmission of Heat.

Specific Conductivity.—The Bureau of Standards, Washington, D. C., has determined the "specific conductivity", *e*, of balsa wood with the following results, expressed in British thermal units per hour, per square foot, per inch of thickness, per degree Fahrenheit of difference in temperature between the surfaces:

e for untreated balsa wood.	*e* for treated balsa wood.
0.394 B. t. u.	0.422 B. t. u.
0.352 "	0.350 "
0.403 "	0.424 "
	0.383 "

The lowest results obtained with both the treated and the untreated wood indicate a "specific conductivity" of 0.350. The higher results in other cases are to be attributed to imperfect specimens,

or to imperfect contact of the heat measuring devices. The Bureau of Standards uses electrical methods in measuring the heat supply and the temperature of the surfaces of the material, in order to eliminate all surface losses. The "specific conductivity" corresponds to e in the equation,

$$H = \frac{e}{x} (\theta - \theta'),$$

where x = the thickness, θ = the temperature of the entering surface, θ' = the temperature of the discharge surface, and H = the heat transmitted.

Heat Transmitted by Melting Ice.—Table 3 gives the results of the writer's investigation of heat transmission by determining the quantity of ice which melted in boxes made of balsa wood and other materials under known differences of temperature measured inside and outside. The results were reduced to British thermal units transmitted per square foot of mean surface between the outside and inside surfaces of each box, per degree of difference of temperature of the air inside and outside, per hour of time, which correspond to the "coefficient" of heat transmission, k, in the equation,

$$H = k (T - T'),$$

in which T = the temperature of the air on the entering side, and T' = the temperature of the air on the discharge side, the other symbols being as before stated. The coefficient, k, differs from the "specific conductivity", e, of the previous equation as it is dependent on the surface as well as the conduction capacity for heat transmission.

Relation Between "Specific Conductivity" and "Coefficient of Heat Transmission."—It is evident that the quantity of heat transmitted through any body is equal to that passing in succession each heat-resisting part. For example, if heat passes through a simple homogeneous body from a higher to a lower temperature, and from the air on one side to that on the other side, it must overcome: (1) the resistance of the entering surface, (2) the resistance of the material composing the body, and (3) the resistance of the surface from which it emerges. The surface resistances, (1) and (3), are overcome by radiation and convection, the interior resistance, (2), by conduction. The surface capacity for transmitting heat may be considered as equal

to the convection capacity plus the radiation capacity. The coefficients of radiation for most materials are known accurately, and may be calculated for different temperature conditions by the application of Stefan's or DuLong and Pettit's Law. The coefficients of convection are not known so accurately, but the values as stated by German engineers appear to give reliable results.

TABLE 3.—Heat Transmission Experiments in Sibley College. Tests Made by Melting Ice.

No. of test.	Thickness of wood, in inches.	British Thermal Units Per Square Foot Per Degree of Difference of Temperature of Air.		Kind of material.
		Per hour k.	Per 24 hours k'.	
1	2	0.121	2.90	Balsa wood, treated.
2	2	0.120	2.89	Balsa wood, treated.
3	2	0.122	2.93	Balsa wood, untreated.
4	2	0.192	4.61	White pine.
5	2	0.102	2.45	Nonpareil cork, extra.
6	2	0.122	2.98	Balsa, single boards, treated.
7	2	0.121	2.90	Balsa, double boards, at right angles.
8	2	0.1194	2.67	Armstrong, XX-cork blocks.
9	3	0.191	2.18	Balsa wood, treated.
10	2	0.1194	2.87	Balsa wood, treated.
11	1	0.199	4.78	Balsa wood, treated.
....	1/16	0.660	15.82	Bare zinc, 1/16 in. thick.

k = coefficient of heat transmission per degree Fahrenheit of difference of temperature of air near the sides, per square foot, per inch of thickness, per hour;
$k' = 24\ k$.

The Bureau of Standards has given the name "specific conductivity" to the quantity of heat conducted, in British thermal units per square foot per hour, per inch of thickness, per degree of difference of temperature of the walls; and this term is used in this paper. This method does not consider surface losses, which vary with conditions. Engineering computations of heat transfer must usually be made by considering the temperatures of the air on the two sides, and require a knowledge of a coefficient of heat transfer per unit of area, per inch in thickness, per degree of difference of temperature of the air on each side, represented herein by k.

The following equations give the relation between "specific conduction", e, and "coefficient of heat transmission", k. They are rational,

and are recognized by French and German engineers as accurate. In the equations which follow,

H = heat transmitted per unit of surface per unit of time;

e = "specific conductivity", or heat transmitted per degree of difference of temperature of the sides of the material, per unit of time, per inch of thickness;

x = thickness;

a_1 = coefficient of surface flow entering the body;

a_0 = " " " " leaving the body;

k = coefficient of total heat transmission per degree of difference of air temperature, per inch of thickness;

$T - T'$ = difference of temperature of air on two sides;

$\theta - \theta'$ = " " " of material on two sides;

$t = T - \theta$ = drop of temperature entering the material;

$t' = \theta' - T'$ = drop of temperature leaving the material.

As the flow of heat is continuous, we have the following expressions, all equal to each other, as representing the flow through a single wall of homogeneous material without air space, per unit of surface, per unit of time:

Flow by conduction through interior . . $H = \frac{e}{x}(\theta - \theta')$ . . . . (1)

Surface flow entering . . . . . $H = a_1(T - \theta)$ . . . (2)

Surface flow leaving . . . . . $H = a_0(\theta' - T')$ . . . (3)

Total heat transmission . . . . . $H = k(T - T')$ . . . (4)

From these we can readily deduce

$$\frac{1}{k} = \frac{1}{a_1} + \frac{1}{a_0} + \frac{x}{e} \quad \ldots\ldots\ldots\ldots\ldots\ldots (5)$$

from which the relations between k and e can be computed, provided the values of a_1 and a_0 are known.

The foregoing equations apply to a single thickness without air space; but it is easy to calculate, by the same process of reasoning, the heat transmission through walls made up of various materials with

or without air spaces. In the case of a wall made up of two different materials with an air space between, we should find by calculation

$$\frac{1}{k} = \frac{1}{a_1} + \frac{x}{e} + \frac{1}{a_0} + \frac{1}{a_1'} + \frac{x}{e'} + \frac{1}{a_0'} \quad \ldots\ldots (6)$$

The temperature of the surface can be readily deduced by transposition in the equations given, thus, from Equation (2),

$$\theta = T - \frac{H}{a_1} \quad \ldots\ldots (7)$$

The surface transmission coefficients have been worked out carefully for numerous cases by Rietschel and other German engineers. These are given by Kinealy,* in English units, as follows:

$$a = c + d + \frac{(40c + 30d)\,t}{1\,000} = c\,(1 + 0.004\,t) + d\,(1 + 0.003\,t)$$

$c = 0.82$ for still air;
$c = 1.03$ for air with moderate velocity;
$c = 1.23$ for air with high velocity.

Values of d.†

Brickwork	0.74	Iron, rusted	0.69
Mortar	0.74	Cast iron, new	0.65
Plaster of Paris	0.74	Sheet iron, polished	0.009
Stone masonry	0.74	Brass, polished	0.053
Wood	0.74	Copper "	0.033
Paper	0.78	Tin "	0.045
Glass, dry	0.60	Zinc "	0.049
Glass, wet surface	1.09	Zinc, dull	0.10

For thick walls and poor conductors, t is so small that it can be neglected. Rietschel gives values of t, for windows, as 36; for brickwork 5 in. thick, as 14; for brickwork 30 in. thick, as 5, for wooden doors, as 2.

Comparison of "Specific Conductivity", e, with "Coefficient of Heat Transmission", k.—The following computation is based on an assumed value of the "specific conductivity" ($e = 0.35$), as determined by the

* "Formulas and Tables for Heating", New York, David Williams Company.

† This table is the same as the Péclét table (see E. Péclét, "Traité de la Chaleur") for coefficients of radiation reduced to English units.

Bureau of Standards, a_0 and a_1 as computed from the preceding equations, and coefficients, which are as follows:

First, For a Single Wall Without Air Space.—

$$a_0 = a_1 = 0.82 + 0.74 + 0.01 = 1.57.$$

These values substituted in the general equation, give

$$\frac{1}{k} = \frac{1}{1.57} + \frac{1}{1.57} + \frac{x}{e}.$$

This, solved for $e = 0.35$ and for values of $x = 1$, 2, and 3, gives the following results:

"Specific conductivity."	Thickness, in inches.	"Coefficient of heat transmission."
$e = 0.35$	$x = 1$	$k = 0.242$
$e = 0.35$	$x = 2$	$k = 0.143$
$e = 0.35$	$x = 3$	$k = 0.1015$

Second, Single Balsa Wall, Ice in Zinc Box; Representing Test Results.—A box of balsa in still air, having thicknesses of 1, 2, and 3 in., respectively; melting ice confined in a dull zinc box, $\frac{1}{64}$ in. thick, the inside of which is in contact with the ice but separated from the balsa wood by an air space. These conditions correspond to those of the ice-melting test of which the results have been stated, and to which Equation (6) applies. For these conditions, $a_0 = 0$, and $\frac{x'}{e'}$ is so small that it is negligible.

Substituting numerical values, we have

$$a_1 = c + d = 0.82 + 0.10 + 0.92$$

$$\frac{1}{k} = \frac{1}{1.57} + \frac{1}{1.57} + \frac{x}{0.35} + \frac{1}{0.92}.$$

This, solved for different values, gives the following results:

"Specific conductivity."	Thickness in inches.	"Coefficient of heat transmission."
$e = 0.35$	$x = 1$	$k = 0.192$
$e = 0.35$	$x = 2$	$k = 0.124$
$e = 0.35$	$x = 3$	$k = 0.091$

Table 4 gives a comparison of the determinations of k by computation (assuming $e = 0.35$) and test.

TABLE 4.—Results with Balsa Wood Box.

"Coefficient of Heat Transmission," k.
"Specific Conduction" e.

Thickness of box, in inches.	By computation, assuming $e = 0.35$ (for 1 hour).	By Test, Melting Ice, in a Zinc Box, Inside Balsa.	
		k for 1 hour.	k' for 24 hours.
1	0.192	0.199	4.78
2	0.124	0.121	2.90
3	0.091	0.091	2.18

Engineers using insulating material for cold storage generally express the heat transmission coefficient on the basis of the heat, in British thermal units transmitted per degree Fahrenheit of difference in temperature of the air on the two sides, through material 1 in. in thickness, and for a period of 24 hours, which corresponds to k' in Table 4. The coefficient for 1-in. material is generally assumed as equal to twice the result obtained in a test of 2-in. material. In practice, only materials 2 and 3 in. in thickness are used, so that the coefficient obtained on such a basis, though not scientific, gives results which are fairly close. On such a basis, the "coefficient of heat transmitted" through balsa wood, per inch of thickness, computed from the Bureau of Standards tests, is 5.98; and, as determined by ice-melting experiments in Sibley College, is 5.80.

DISCUSSION

Mr. Lundin.

A. P. Lundin,* Esq.—The speaker's attention was drawn to this wood many years ago, during voyages to tropical countries, *viz.*, Central and South America, Central Africa, and also India, in all of which the same species can be found; and he first remarked it when a number of natives came floating down a river on a raft made up of balsa logs. The logs were covered more or less with bark, and where this was chipped away there still remained the hard surface which is found on the outside of balsa logs, under the bark; the ends were covered with tar, or some waxy substance.

The natives, particularly in Central and South America, use such rafts to float their products to the sea coast, and seldom use them more than once, for one reason, because it would be difficult to bring the rafts up against the stream, and because the wood absorbs water very readily and the raft is more or less water-logged after arriving. Of course, in solid logs, with the ends closed up, the absorption is not so rapid as when balsa is freed from bark and cut up in planks.

Later, when engaged in the life-saving equipment business, it was brought to the speaker's attention that some crude attempts had been made to use balsa in life preservers. On taking over a boat shop in Long Island City, a quantity of balsa was found there, and, on inquiry as to its purpose, it was learned that experiments had been made with it in life-belt manufacture, but that the wood absorbed water so rapidly that the belts had to be made two or three times as large as the ordinary cork life-belt to assure the required buoyancy. Subsequently, chemical experts were put to work to devise methods for making the wood non-absorbent.

First, painting was tried, but, owing to the peculiar nature of the material, the paint was rapidly absorbed, and coating it over and over meant having just as much paint as wood, which added greatly to the weight. Then varnishing was tried, but, owing to the moisture left inside, the varnish cracked and blistered off.

Next, several mixtures of paraffin, asphaltum, gilsonite, etc., were tried, which gave a fairly good outside coating, penetrating about $\frac{1}{4}$ in. on the ends and about $\frac{1}{16}$ in. on the sides, and the problem was apparently solved. However, before long it became evident that, owing to the cellular structure of balsa, which is mostly pith, and the great quantity of moisture sealed up in the wood by the impervious surface treatment, dry-rot developed even sooner than in the untreated balsa.

* New York City.

Mr. Lundin.

Just at the time these difficulties became apparent, Col. Marr's water-proofing process was brought to the speaker's attention, and after numerous experiments with the new method, it was successfully and practically applied.

The United States Government tried out balsa life-preservers, life-buoys, etc., as compared with the cork articles, for a period of 49 days (24 hours per day), at the end of which period the cork preserver had lost all its buoyancy and the balsa preserver still retained the buoyancy stipulated in the Government requirements.

A few years ago, while working on the buoyant-material proposition, it was considered that, as balsa, owing to its peculiar structure, was so advantageous for use in buoyancy products, it might also be adaptable for insulation purposes, and accordingly experiments in that direction were begun. The first ice-box made of the new material was on the speaker's motor boat, and the results were surprising. All during the hot summer weather, ice was put in the box on Friday or Saturday, and on the following Friday or Saturday the temperature in the box would still be quite low and some ice was still left in the box.

Naturally, all first work in the line of balsa insulation was more or less crude, and the importance of scientific investigation was soon realized.

It was particularly fortunate that Professor Carpenter became interested in this material. The speaker well remembers that when he first spoke to him about this wood, and stated that it was all pith and no fiber, he and the gentlemen in his company looked very skeptical. However, he was sufficiently interested to visit the Welin plant, intending to remain there half an hour, but he spent practically a whole day, and when last seen on that occasion, he had all his pockets full of balsa, and he has been steadily devoted to the investigation ever since.

The speaker does not pretend to be an expert on insulation or non-conductivity, but looks at this material from a practical rather than a scientific viewpoint.

The principal feature in insulation material is, of course, that it must be a good non-conductor, but no doubt in the future engineers will also consider structural strength, and the possibility of making up complete homogeneous units will also be considered in judging the efficiency and value of insulating material, particularly where it is to be used in making ice-boxes and as insulation for buildings, in ships, and in railroad cars.

The principal consideration is a commercial one; in other words, good engineers always try to obtain the highest total efficiency, and commercial men want it and are willing to pay for it.

In shipping, for instance, it is known that almost any ship can be insured if built to certain requirements, such as Lloyds', and it

Mr. Lundin.

can be insured at the lowest rates (except, of course, in time of war). All cargo on such ships can also be insured at reasonable rates, except one class, and that is perishable food stuffs carried in refrigerator compartments, which are always carried at the shipper's or consignee's risk. Neither the steamship companies nor the insurance organizations will place insurance on such goods, except for the event of total disaster to the ship. Why? Not because the science of refrigeration has not kept pace with the science of naval architecture, but because of the fact that if a break-down occurs in the refrigerating machinery, the cargo will spoil, as the insulation is not reliable enough to keep the temperature sufficiently low until the machinery can be repaired and the system put to work again.

The speaker had an experience of this kind, many years ago, when on board a meat ship running from Australia to London. The machinery broke down while running through the Red Sea, and more than half the cargo spoiled in less than 24 hours.

A few years ago, in a meat market in Seattle, a new refrigerating plant, which had what might be called ordinary commercial insulating material, broke down, and before new parts could be obtained to replace the broken ones, and the machinery put in working order again, the damage to the meat in the market amounted to more than the cost of the refrigerating plant.

Therefore, when it comes to efficiency in insulation, there is still room for improvement. Perfection cannot be attained, but balsa wood will surely help to make insulation perfect, as it is not only a very efficient non-conductor, but has sufficient structural strength, and, through its physical properties, permits of constructing units with practically unbroken insulated surface walls.

A small ice-box, or pony refrigerator, made by the Welin Company, is of balsa wood, 2 in. thick, about 36 in. long, 21 in. wide, and 22 in. deep, and weighs about 30 lb. Such a box could not be made up of any other known insulating material. It is strong enough to stand severe jars, and a man could jump on it without straining it unduly. Of course, other woods will stand more rough usage than balsa, but, to meet this factor of additional strength, particularly on the surface, paneling made from the bark and waste of the wood is applied to the outside.

A small container, on the order of a thermos bottle, but in the form of a box made of 1-in. material, has a capacity of about 1 cu. ft. and weighs 6 lb. When going on an automobile trip, or something of that sort, the "lunch" can be placed in such a container—whether it is to be kept hot or cold. If it is wanted cold, a little ice should be put in to keep it so.

Butter has been sent all the way from Virginia to Southern California in such boxes, at an average outside temperature of 82°,

and the trip took 8 days, by the slowest route. Yet, when the boxes arrived at Los Angeles, the butter was still hard and frozen. Mr. Lundin.

Even if balsa had 30% less efficiency as a non-conductor, the speaker believes that it would meet a very common requirement for insulation, not satisfied by another material lacking in structural strength, which makes it possible to eliminate all leakage of heat through imperfect joints, or by use of cement or nails which may be classified as good conductors. Other insulating materials are almost entirely limited to use as a lining for a structure built of good conducting materials.

The speaker has a dream of perfect refrigerated transportation and conservation which may be of interest: It is growing more and more expensive to live in America and in many other countries. Food seems to be getting more scarce and more expensive all the time; heat in the winter and cold in the summer are rising in price, in short, the cost of every daily need is "going up."

Much of the increase in prices is due to waste, and, if this waste can be eliminated, it stands to reason that prices will go down. If foodstuffs could be transported, stored, and kept in first-class condition, not only the tremendous waste which exists to-day would be done away with, but food would be purer and would retain its full nutritive values, and this is very important, as there is no doubt that cold-storage food will have to be used to a large extent.

By instituting a continuous chain of proper and effective cold-storage and transportation facilities, which would insure the safe delivery of foodstuffs with a minimum of deterioration and a minimum of waste, some of the chief causes of the high cost of living would be removed, and this balsa insulation system could be spread, not only over the Western Continent, but all over the world. Foodstuffs would travel from producer to consumer in balsa containers—parcel post boxes, pony refrigerators, automobiles, railroad cars, ships—to be stored in balsa-walled storage warehouses, and, when delivered, would be put away in balsa house refrigerators.

The scope of this enterprise is very great. Insurance and guaranties would be furnished on such shipments, for with such highly improved methods to lessen the possible loss, insurance companies would find it to their interest to establish a standard specification on which basis the risk on perishable foodstuffs in transit could be covered.

The time is not far off when it will pay every architect to line his buildings with 1 or 2 in. of balsa wood, not only as an insulation lining but as a finish, instead of plaster and such coatings, which crack and fall down. Very attractive effects can be obtained with paneling in the interior of buildings.

There are a thousand and one uses for balsa, and in the future new ones will continually develop. Thanks and grateful acknowledg-

Mr. Lundin. ment are due to Professor Carpenter for the earnest scientific work he has done in investigating this wood.

Mr. Cox. LEONARD M. COX,* M. AM. SOC. C. E.—Although he can add nothing of professional value to the discussion of this excellent paper, the speaker feels a curious interest in the insulating properties of balsa wood. Some 15 years ago, it was his fortune to serve as Chief of Public Works at the Island of Guam, and among a number of more or less serious problems which confronted the pioneers of that day, was one which was never solved in an entirely satisfactory manner with the materials available. The cold storage room of the miniature ice plant served fairly well for caribou and American beef, but invariably caused the loss of fully half the mutton and poultry begged from the army transports on their monthly visits. From a first reading of the paper, it seemed at least possible that a very common Guam tree was akin to, if not identical with, the balsa tree, and that the remedy for our cold storage troubles had been near at hand though unrecognized. Personal notes consulted since the presentation of the paper before the Society establish the fact that the Guam tree, known locally as pago (*Hibiscus tiliaceus*), bears slight resemblance to the balsa tree, except as regards its comparative lightness and structure, as indicated by cut surfaces. Its weight per cubic foot is 24 lb.; it has considerable bending strength; is of about the color of Flemish oak; and rarely grows to a size exceeding 6 in. in diameter.

The reference in the paper to the Marr wood-preserving process also interests the speaker for the reason that while he was on duty at the Norfolk Navy Yard in April, 1914, a number of experimental timbers, running from 8 by 8 in. to 10 by 10 in., were impregnated with the Marr paraffin, in which there was incorporated finely divided silica. These pieces were suspended half in water and half in air, beneath one of the Yard wharves. At the same time six pieces, 4 by 6 in. and 4 by 4 in., were impregnated with the paraffin without silica, and buried to half their lengths in the ground. These test pieces were examined on July 15th, 1915, and found to be in excellent condition. They have also been examined during the latter part of June, 1916, and show no evidences of deterioration. The specimens which had been placed in water had a small quantity of shells and vegetable growth attached to their surfaces, all of which promptly fell off when the timbers were struck. On cutting into the wood, it was found that the interior was perfectly bright, and exhibited the distinctive red color of the process. The pieces in the ground show no signs of rot or age. Of course, tests on such a small scale cannot be considered determinative, and it is hoped that other engineers will give the Profession the benefit of any experience they may have had with this process.

* Brooklyn, N. Y.

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Paper No. 1383

SUGGESTED CHANGES AND EXTENSION OF THE UNITED STATES WEATHER BUREAU SERVICE IN CALIFORNIA

REPORT OF A COMMITTEE OF THE SOUTHERN CALIFORNIA ASSOCIATION OF MEMBERS.*

BY GEORGE S. BINCKLEY, M. AM. SOC. C. E.,
AND CHARLES H. LEE, ASSOC. M. AM. SOC. C. E.

WITH DISCUSSION BY MESSRS. N. C. GROVER, WILLIAM S. POST, CHARLES T. LEEDS, FRED. H. TIBBETTS, J. B. LIPPINCOTT, AND GEORGE S. BINCKLEY AND CHARLES H. LEE.

The Committee, appointed by the President of the Association on April 22d, 1914, to outline a plan for strengthening and extending the mountain climatological work of the United States Weather Bureau in California, so as to meet more widely and fully the needs of the industries and communities of the State relative to water supply, respectfully submits the following report.

Climatology is defined as that subdivision of meteorology which treats of the physical condition of the atmosphere during an extended period of time. Knowledge as to the climate of a place is obtained by a study of its continuous weather records—atmospheric pressure, temperature, rainfall, snowfall, direction and velocity of the wind, relative humidity, and evaporation—for a long period of years. The general relation of climatic factors to run-off and stream flow, as cause and effect, is so obvious, even to the layman, that no discussion is necessary.

* This report was presented to the Southern California Association of Members, Am. Soc. C. E., at its meeting of October 14th, 1914, and received the endorsement of the Association.

Throughout much of California and the Southwest the important streams have their sources in the mountain ranges bordering the valleys, and receive an almost negligible contribution from the valleys. In many districts arid valleys lie adjacent to mountain ranges receiving an annual precipitation of from 20 to 30 in. In such regions the gathering of climatic data, as a basis for the study of stream flow and water supply, should be carried on in mountains within the productive portions of drainage areas. This fact has been generally recognized throughout the West by irrigators, engineers, and some local officers of the United States Weather Bureau, ever since the commencement of irrigation development on a large scale.

The intimate relation which mountain precipitation, through stream flow, bears to the development and settlement of California is well shown by consideration of the communities and industries depending on reliable water supply. Agriculture, the most important industry of the State, to reach its greatest success, must generally be accompanied by irrigation. This has been fully demonstrated in Southern California and the San Joaquin Valley, and its truth is now recognized in the central and northern part of the State. The agriculturist, therefore, is fully as much interested in mountain climatology as related to water supply as he is in the atmospheric conditions immediately surrounding his crops. Hydro-electric power is rapidly taking a leading place throughout the State in manufacturing, transportation, and agriculture, as well as in supplying the physical needs of the inhabitants for light and heat. The reliability of the supply is largely dependent on stream flow, so that here, again, the average citizen is interested in mountain precipitation. Municipalities throughout the State depend for their very existence on mountain stream flow, for, if they do not derive their water supply directly from mountain run-off, as in the new Los Angeles Aqueduct system and the proposed Hetch Hetchy development, their water is pumped from underground reservoirs which, in turn, depend on precipitation and stream flow for replenishment. Finally, there is the subject of flood control and prevention which affects more or less the whole State and requires, to the fullest degree, definite and extended knowledge of mountain climatic data for its correct solution. In connection with this there is also the navigation of inland streams to be considered in the Great Central Valley of the State. The many

interests which would benefit by the collection, co-ordination, and publication of climatic data from the mountains are thus apparent.

The practical usefulness of climatic data is in forming a basis for the forecast of future stream flow variation. It is on the latter

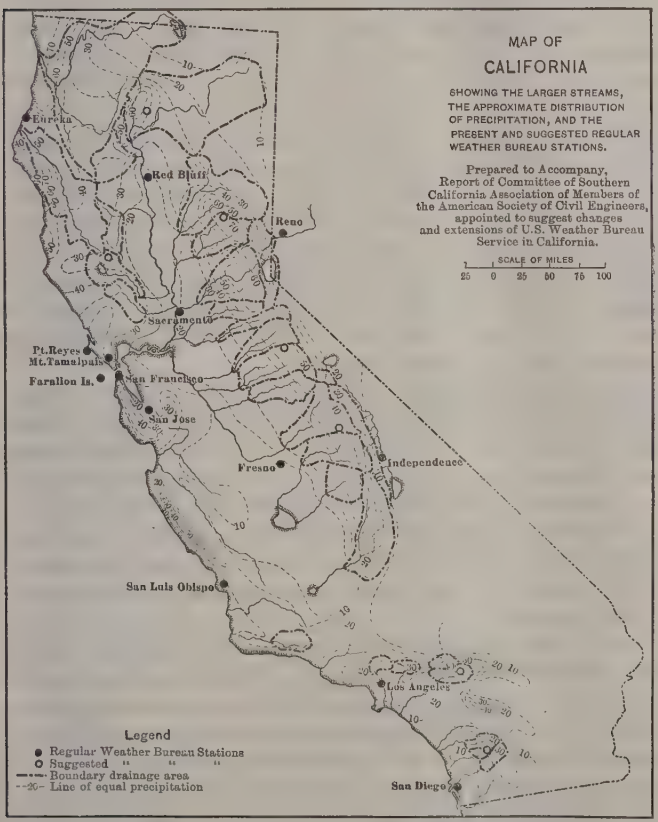
MAP OF
CALIFORNIA
SHOWING THE LARGER STREAMS, THE APPROXIMATE DISTRIBUTION OF PRECIPITATION, AND THE PRESENT AND SUGGESTED REGULAR WEATHER BUREAU STATIONS.
Prepared to Accompany, Report of Committee of Southern California Association of Members of the American Society of Civil Engineers, appointed to suggest changes and extensions of U.S. Weather Bureau Service in California.
SCALE OF MILES
25 0 25 50 75 100
Eureka
Red Bluff
Reno
Sacramento
Pt. Reyes
Mt. Tamalpais
Farallon Is.
San Francisco
San Jose
Fresno
Independence
San Luis Obispo
Los Angeles
San Diego
Legend
Regular Weather Bureau Stations
Suggested " " "
Boundary drainage area
Line of equal precipitation

FIG. 1.

information that the magnitude or practicability of irrigation projects, hydro-electric power systems, municipal water developments, flood-protection works, or any other large engineering work, depending on

water supply or stream flow, is based. Such data have also great value in the design of engineering structures for the control of water, such as dams, spillways, head-works, bridges, levees, etc. There is still another field that is beginning to be recognized, namely, in the business management of projects depending wholly or in part on snow storage. Where snow storage is the only source of supply, as is the case in many irrigation and small power projects, a predetermined knowledge of the season's supply would be of great value in making contracts for the delivery of water or power. Where the snow storage is supplemented by other sources, a previous knowledge of the quantity available makes possible a more complete economic planning for the development of an auxiliary supply. The work of the United States Weather Bureau in the Wasatch Mountains, Utah, has shown the practicability of making snow surveys preceding the period of melting which are sufficiently accurate to form the basis of contracts, between the irrigation companies and the irrigators, for the season's delivery of water in the valleys.* Such data from the Sierra Nevada streams of California would be of State-wide value.

It can be said, therefore, that comprehensive, accurate, and properly correlated data regarding mountain precipitation and related climatic data, such as temperature, wind movement, barometric pressure, humidity, and evaporation, are of the utmost importance in the permanent and economically efficient development of the State.

The extent to which such data are made available by the United States Weather Bureau in California will now be considered, first as to the geographic and vertical distribution and then as to the character of the observations.

The Bureau maintains stations of two classes: regular stations, fully equipped with standard recording instruments, in charge of trained observers; and co-operative stations, with incomplete instrumental equipment, observed without compensation, by public-spirited citizens. There are 13 regular stations in California, of which 6 are on the sea coast, 2 on the coastal plain, 4 in interior valleys, and 1 at Independence, Inyo County, at the western edge of the Great Basin, in an arid region. There are no stations in mountainous regions, or in fact in any region from which the local precipitation and run-off will ever be an important source of water supply. There are listed in

* *Engineering News*, May 29th, 1913, p. 1110.

the California Section of the Climatological Service of the Weather Bureau, Annual Summary for 1913, 192 co-operative stations, of which 17 (9%) are within 25 miles of the coast; 104 (54%) are in interior valleys and foot-hills; 16 (8%) are in the Great Basin and desert; and 55 (29%) are in regions of broken topography where the elevation exceeds 2 000 ft. Of the latter, 4 are above 6 000 ft. elevation, 7 are above 5 000 ft., and 21 are above 4 000 ft. The vertical limits of the most copious mountain precipitation in California are between 4 000 and 6 000 ft. elevation. Of the 21 stations above 4 000 ft. elevation, 13 are in the Sierra Nevada Range, the remainder being well scattered over the State. Of the 13 in the Sierra Nevada Mountains, 10 are concentrated in the group of counties: Butte, Nevada, Placer, Plumas, Sierra, and Alpine, which comprise only one-fifth of the length of the range. The other 80% of this highly productive drainage area has only 3 stations above 4 000 ft. elevation. The important area tributary to San Joaquin, Kings, Kaweah, Kern, and Owens Rivers has only one station, namely at Bishop Creek, Inyo County, at an elevation of 8 500 ft.

The character of the climatological observations made by the Bureau will be considered: first, as to numerical climatic data obtained; second, as to location and exposure of instruments; and third, as to qualifications of observers.

The following data are obtained at regular stations:

(1) Continuous air temperature records;
(2) Wet and dry bulb thermometer observations for relative humidity at 5 A. M. and 5 P. M.;
(3) Continuous precipitation records;
(4) Clearness of atmosphere;
(5) Continuous record of direction and velocity of wind;
(6) Continuous record of barometric pressure;
(7) Occasional water evaporation records;
(8) Miscellaneous observations.

Such data are all useful to the engineer as well as the climatologist, and if obtained under representative local conditions are of great value.

At co-operative stations served by volunteer observers the following data are obtained:

(1) Daily readings of maximum and minimum thermometers;

(2) Total precipitation each 24 hours as caught and observed in standard 8-in. gauges (snow catch usually computed as equivalent water in ratio 10 to 1), depth of snowfall in 24 hours measured on the ground, total depth of snow on ground at time of observation;

(3) Miscellaneous phenomena, such as occurrence of frost, coronas, thunderstorms, tornadoes, auroras, general phenomena of climate, and general character of the day.

The method of snow measurement prescribed at co-operative stations is rather unsatisfactory, both in catching and in reducing to equivalent water. Great improvement would result from the substitution of snow bins for the 8-in. rain gauges. It is a simple matter to cut and weigh cores from the snow on a bin floor. Such a procedure would eliminate the present elements of inaccuracy: first, as to the effect on snow catch of wind currents about the mouth of the 8-in. gauge; second, as to judgment of observer in ascertaining snow depth; and third, in the use of the approximate ratios of fallen snow depth to equivalent water.

The location of instruments at regular stations in large towns and cities (9 out of 13 stations) is on the top of high office buildings. This has a tendency to detract from the practical value of temperature, precipitation, and wind observations. As regards temperature, it is a frequently observed fact in Southern California that on winter mornings, when low growing crops are suffering from freezing temperatures, the official record will show much milder temperature. This can be explained, first, from the fact that the record is kept in an upper and warmer air stratum, and second, by convection and radiation from the steam-heated building on which the thermometer shelter is placed. Then, as regards precipitation and wind, tall buildings cause vertical and cross-currents and eddies, and these obviously affect the precipitation catch as well as the anemometer record. At the regular stations in small towns and at isolated points the location is usually on the open ground, and far better exposures are secured. At co-operative stations the thermometer shelter and rain gauge are usually on the ground, but the gauge is often very poorly placed with respect to obstructing objects such as trees, outbuildings, or houses.

The observers at regular stations are trained men who usually make a life study of meteorology, and are accurate and systematic in their

work. They are essentially indoor office men, however, and are not encouraged by the Washington office to get out into the field, observe conditions on the ground, and keep in close touch with local conditions and the work of co-operative observers. There is no such body of active, efficient, field observers, for instance, as is maintained by the Water Resources Branch of the United States Geological Survey. As a result of this policy, the Weather Bureau has in many respects failed to meet the plain needs of western communities and provide data of uniform reliability.

The co-operative observers are private citizens, without training in systematic and accurate observation, who volunteer their services, and receive no compensation. They place and observe rain gauges without official supervision or instruction. Instruments are often not understood, or are out of repair, readings are occasionally omitted, and the general character of the observations is not by any means up to a first-class standard. The obvious remedy for these defects is frequent personal supervision by trained men from the district central office. Nevertheless, the co-operative observers have filled a great need, and the Weather Bureau is to be commended for its activity in finding public-spirited citizens who are willing to undertake such work, and for the rapidity with which these records are made available to the public, and the convenient manner in which they are presented.

Summarizing the work of the Weather Bureau in California, as regards the gathering of mountain climatic data, it appears:

First.—That no regular stations are maintained within the productive drainage area of any important stream of the State, nor are any systematic scientific studies made of the subject;

Second.—That of the 55 co-operative stations maintained in mountainous regions, 21 are above 4 000 ft. elevation, of which only 13 are in the Sierra Nevada Range, and 10 of the 13 are concentrated within about 20% of the area of the west slope;

Third.—That the co-operative stations are not equipped with instruments or men capable of gathering intelligently the data necessary for a comprehensive scientific study of mountain climatology as related to stream flow;

Fourth.—That the Weather Bureau does not maintain a corps of trained field observers capable of supplementing and keeping in touch with the co-operative observers;

Fifth.—That the placing of temperature, precipitation, and wind recording instruments on high buildings is practically without value to the agriculturist or engineer who attempts to use the data thus obtained.

In great contrast with the lack of regular stations in the mountains is their superabundance in the populated districts and along the coast. For example, within 50 miles of San Francisco there are 5 regular stations, San Francisco, San José, Point Reyes, Tamalpais, and Farallon. In the Great Valley there are 3 regular stations, Red Bluff, Sacramento, and Fresno; and along the south coast there are also 3 regular stations, San Luis Obispo, Los Angeles, and San Diego. At Independence, in a desert region possessing the characteristics of the Great Basin, there is maintained a fully equipped station which, because of its location, can have no practical value in solving the water supply problems of the State, unless supported by a station in the High Sierra to the west. Hence, there is evident a great inequality and inconsistency in the present distribution of regular Weather Bureau stations.

The Committee, after careful consideration of the facts as just stated and a study of the needs of the situation, presents the following outline as a working basis for extending the usefulness of the Weather Bureau:

I.—Establish at least 6 regular stations in the most productive mountain drainage areas, fully equipped with self-recording instruments and in charge of trained observers fitted for active outdoor field work. These stations should be, not only points at which climatologic data are collected, but also the headquarters for field observers having in charge local co-operative observers, and carrying on widely extended climatic surveys and special studies. If lack of funds prohibits the establishment of new stations, there is no reason why most of the existing stations could not be moved. Such action would be rather hard on certain of the Weather Bureau men, but would be of great benefit to the people of the State as a whole, and tend to distribute the benefits of the service more equitably. The locations of such stations suggested by the Committee are as follows:

1.—In the drainage area of McCloud or Lower Pitt Rivers;

2.—In the drainage area of the Feather or Yuba Rivers;

3.—In the drainage area of Stanislaus or Tuolumne Rivers;
4.—In the drainage area of San Joaquin or Kings Rivers;
5.—In the drainage area of Cache Creek above Clear Lake;
6.—In the drainage area of the Santa Ana River;
7.—In the drainage area of San Diego or San Luis Rey Rivers.

The conditions which should govern the location of these stations are: accessibility by wagon road or railroad from populated districts; accessibility in winter by skis or other means of travel for purposes of reading snow gauges and making snow surveys at points several days' journey distant; topographic features such as to insure the representative character of the station records; they should be in the zone of maximum rather than minimum precipitation for the drainage area; and, generally, should be where the facilities for the study of climatology as related to local water supply would be the best.

II.—Increase the number of co-operative stations in mountain areas now sparsely covered or not covered at all. In uninhabited areas, where neither mining settlements nor forest ranger stations are available, establish rain and snow gauges capable of holding a catch for an extended period.

III.—Organize a corps of trained field observers whose duties it will be to supervise personally the work of co-operative observers, and make special field studies, such as listed under IV.

IV.—Make special studies as follows, in as wide a territory as possible:

1.—Annual snow surveys in the drainage areas of Sierra Nevada streams;
2.—Snow movement subsequent to precipitation;
3.—The relation of temperature, forest trees, drifting, etc., to snow melting;
4.—The detailed relation between topography and precipitation;
5.—Water evaporation from lakes or reservoirs;
6.—Evaporation from snow;
7.—Establish, maintain, and observe snow and rain gauges at isolated points which can be reached only at long intervals;
8.—Any other investigations of practical value in the solution of the water-supply and flood-protection problems of the State.

The Committee has been frank in its criticism of the apparent fact that the principal meteorological observatories have generally been located rather more with reference to the comforts of a city than to secure the greatest efficiency and collect the most valuable data. This criticism, however, is not unfriendly. The staff of the Weather Bureau is uniformly courteous and obliging in furnishing whatever data are available; but, that these data have not a higher standard of reliability and greater practical value is due, the Committee believes, more to a lack of recognition of the great service that could and should be performed by the Weather Bureau, than to any reluctance on the part of those at its head to extend the scope and improve the value of its work.

In conclusion, the Committee does not hesitate to express the opinion that the questions involved in an improvement in the character of service rendered by the Weather Bureau may be discussed by engineers with the most complete freedom, and suggestions may be made without the slightest diffidence. Without in any way reflecting on the unquestioned scientific attainments of the officials of the Bureau, it is safe to say that the engineer who has to deal with the practical questions of water utilization is better qualified to indicate the source and character of meteorological data which will have the greatest real value, than the detached and abstract observer of meteorological phenomena.

The conclusions arrived at by the Committee are the result of mature consideration, and are now laid respectfully before the Association.

DISCUSSION

N. C. Grover,* M. Am. Soc. C. E. (by letter).—The writer is in hearty accord with the suggestions made in this report for rendering the climatological work of the United States Weather Bureau of greater practical value to the communities and industries of California. The desirability of collecting systematic records of temperature, precipitation, accumulation of snow and its equivalent in water, has long been recognized by Weather Bureau officials, as well as by others who need records of that character. The difficulty, as the writer understands it, however, is in devising instruments that will record correctly the phenomena desired, without daily attention, for it is unusual to be able to obtain reliable observers at high elevations in the mountains. Without such resident observers, or instruments that can be relied on to record the data without attention for several weeks, the cost of collecting the records becomes practically prohibitive. There is no doubt, however, that the officials of the Weather Bureau will gradually develop methods by which satisfactory records may be collected at high altitudes. Mr. Grover.

The following statements (pages 162 and 163) seem to be likely to be misinterpreted:

"In such regions the gathering of climatic data, as a basis for the study of stream flow and water supply, should be carried on in mountains within the productive portions of drainage areas.

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"The practical usefulness of climatic data is in forming a basis for the forecast of future stream flow variation. It is on the latter information that the magnitude or practicability of irrigation projects, hydro-electric power systems, municipal water developments, flood-protection works, or any other large engineering work, depending on water supply or stream flow, is based. Such data have also great value in the design of engineering structures for the control of water, such as dams, spillways, head-works, bridges, levees, etc."

Though the statements are not definite in this respect, the reader is left to infer that climatic data collected at properly located mountain stations will yield all the information as to water supply and its variations needed for the design of the important works mentioned. The dangers attending the use of estimates of water supply based on records of precipitation have been forcefully demonstrated by the many failures of hydraulic works of all classes. Such failures have been especially numerous in the irrigated areas of the West, and should serve as an object lesson not to be forgotten, but, unfortunately, many engineers appear never to have learned the lesson at all. Therefore, the inference that might be drawn from the report is particularly fal-

* Washington, D. C.

Mr. Grover. lacious, in view of the reliance that many engineers, both in the East and the West, continue to place on estimates based on records of precipitation, even though it has been demonstrated again and again that such estimates are not only not dependable, but positively misleading.

Mr. Post. WILLIAM S. POST,* ASSOC. M. AM. SOC. C. E. (by letter).—There is one feature of the Weather Bureau Service which only has to be stated to be recognized, namely, that this Bureau, as a matter of fact, has a monopoly of publication. It is, of course, a national organization, and perhaps cannot recognize local conditions to too great an extent, but the writer wishes to emphasize publication as an important matter, in addition to the recommendations of the Committee. There are numerous rainfall records in California, and presumably in other States, which are in the hands of private parties and cannot qualify as official. These records are of the utmost importance because they frequently go back as far as the early Seventies, and are fast disappearing with the dying out of the original observers. We have obtained or are maintaining some 60 stations alone in San Diego County which have no official means of publication. The writer would suggest that the Weather Bureau consent to publish all records in the State, in a volume devoted simply to rainfall, to be contributed from all known sources of information, the responsibility for the record being stated. In this way an extremely important source of information will be preserved.

In reference to this matter, the writer, alone and single-handed, some two years ago, attempted to convince the head of the Weather Bureau that the form of publication, in so far as it gives long lists of "normals", is a needless incumbrance of the present reports. To state it differently, what is required is not the mathematical interpretation of differential variations, etc., but a plain tabulation of the observed data. The first result of this, to the great relief of some engineers, would be a publication, 8½ by 11 in., which is a standard filing size, and the increase of the size of the type from a microscopic nonpareil to a good book type. It may be possible to convince a physical scientist that these normals have some significance, but most hydraulic engineers will agree that they are an unmitigated nuisance in the present publications.

The writer has suggested to one of the members of the Committee that, in case it is not possible to co-operate fully with the Weather Bureau in this matter of publication, the Society itself or the Local Association make permanent the present committee and undertake the publication of the meteorology of California, combining as contributors all those who can furnish any data.

* San Diego, Cal.

Mr. Post.

In hydraulic calculations, certain stations have become in the nature of base stations, and are very valuable for the purpose of restoring broken records or years of no record in the same region. For this reason it is a real loss to have a long-term station discontinued, merely, as it would seem, because the voluntary observer has moved and the Weather Bureau official has neither the money for traveling expenses to inspect and secure another observer, nor funds to hire a paid observer. The writer has recently expressed his regret at the discontinuance of the splendid record at Fort Wingate, in Arizona, to the extremely intelligent and competent Director of the Weather Bureau at Santa Fé, N. Mex., and was told that his hands were absolutely tied in the matter, that he could only secure a voluntary observer through correspondence, and had attempted to do so. That is a case where, because of the lack of an efficient field inspector, who could discover observers, a record which dates back to 1867 has been broken for about 3 years. Numerous other instances could be brought up.

The suggestion is also made that the Weather Bureau take up the study of local drainage areas for a few years, in order to determine the local shapes of the isohyetose lines once and for all in that area, and thereafter proceed with only one base station in that drainage area. The writer has succeeded in drawing such curves on certain drainage areas in Southern California by the establishment of some fifty rain gauges. In some cases there are voluntary observers and in others $1.00 per month is paid. About 15 gauges are examined by one hydrographer, who is also a good rider and can crank a machine. From a comparison of these curves, now that they have been established, it is obvious that a very considerable reduction in these stations can be made.

Mr. Leeds.

Charles T. Leeds,* M. Am. Soc. C. E. (by letter).—The genesis of this paper was an address before the Southern California Association of Members, on April 8th, 1914, by Dr. Ford A. Carpenter, Local Forecaster of the U. S. Weather Bureau, which paper has since been published by the Weather Bureau under the title of "Flood Studies at Los Angeles". The discussion of this paper at that meeting developed the fact that the Weather Bureau, due to lack of funds and observers, is unable to establish as many co-operative stations as it would like to do. Dr. Carpenter stated that, if instruments could be furnished from other sources, he felt sure the Chief of the Weather Bureau and the Section Director at San Francisco would be glad, through the co-operation of the Forest Service, to assist in making the observations. He also stated that as soon as these extensions were authorized he would be glad to instruct the observers and collate all data.

Messrs. Binckley and Lee, therefore, were appointed as a Com-

* Los Angeles, Cal.

Mr. Leeds.

mittee, with the original purpose of investigating the possibility of obtaining the necessary rain gauges and of recommending advisable locations therefor. J. B. Lippincott, M. Am. Soc. C. E., then President of the Association, later instructed this Committee to broaden the scope of the report, as has been done.

The authors deserve commendation for the constructive recommendations which they have made. The Weather Bureau has been of great service in years past, and is now still further extending its usefulness. It is hoped that what criticisms have been made in the paper will be taken as meant only in a spirit of helpfulness.

Many who have not been in the Government service fail to appreciate the limitations imposed by legislative restrictions and lack of appropriations. These limitations should be borne in mind when considering such matters. It is hoped that public opinion may be so awakened as to lead Congress to remove these restrictions and provide adequate funds to permit the Weather Bureau to render to the country the greater service which it should be enabled to perform.

The authors have done well to state in their opening paragraph their purpose "to outline a plan for strengthening and extending the mountain climatological work of the United States Weather Bureau in California, so as to meet more widely and fully the needs of the industries and communities of the State relative to water supply". The paper deals only with this phase of the work of the U. S. Weather Bureau. It may lead to a better understanding of the whole matter to trace briefly here the development of the Weather Bureau, and state the duties now assigned to that Bureau by law.

The service now performed by the Weather Bureau was originally organized and carried on by the Signal Corps of the U. S. Army, and included only meteorological observations at military stations for the prediction of storms. To this was added in 1870 the duty of rendering reports relative to the stage of water in rivers. This service was primarily for the benefit of navigation on the ocean and the Great Lakes, and later included the interior districts and the great rivers. By Act of Congress, on October 1st, 1890, these duties were turned over to the present U. S. Weather Bureau of the Department of Agriculture. The duties then assigned to it were as follows:

"Forecasting the weather; the issue of storm warnings; the display of weather and flood signals for the benefit of agriculture, commerce and navigation; the gauging and reporting of rivers; the maintenance and operation of sea coast telegraph lines and the collection and transmission of marine intelligence for the benefit of commerce and navigation; the reporting of temperature and rainfall conditions for the cotton interests; the display of frost, cold wave and other signals; the distribution of meteorological information in the interest

of agriculture and commerce; the taking of such meteorological observations as may be necessary to establish and record the climatic conditions of the United States or are essential for the proper execution of the foregoing duties." Mr. Leeds.

It is regrettable that no mention is made by the authors of the storm warnings, sent out by the Weather Bureau, resulting in the prevention of untold disaster and loss of life. Nor is there any mention of the excellent system of frost prediction, such as has been developed in Southern California. Evaporation studies are urged (and wisely), but the implication is that none has been made. The authors have apparently overlooked the evaporation studies made by the Weather Bureau at Lake Tahoe and the Salton Sea. No mention is made of the careful and scientific studies, by the Aerial Research Department, of the upper air currents, made not long ago at Avalon, Cal.

The authors seem also to have overlooked the fact that the typical action of a storm center on this continent is to enter from the North Pacific, moving thence south-eastward and then north-eastward across the continent, and passing out into the Atlantic Ocean from New England or the St. Lawrence Basin. The most severe storms, however, enter through Southern California and proceed eastward and north-eastward across the continent. For the prediction of storms in the central and eastern part of the United States, therefore, it is essential that evidence of an atmospheric change or disturbance should be gathered as early in its course as practicable, and its direction and rate of progress must be carefully traced from place to place. It is doubtless to facilitate this purpose that so many regular stations are scattered along the coast. It must also be borne in mind that the regular weather bureau station, being a central distributing station for climatic information, must be situated so as to facilitate this collection and distribution of data. This of necessity requires that the regular stations shall be in centers of population.

The writer, as a hydraulic engineer, realizes fully the need of the information desired, and is thoroughly in sympathy with all extensions suggested. He is unable, however, to agree with the following statement on page 168:

"If lack of funds prohibits the establishment of new stations, there is no reason why most of the existing stations could not be moved. Such action would be rather hard on certain of the Weather Bureau men, but would be of great benefit to the people of the State as a whole, and tend to distribute the benefits of the service more equitably."

On the contrary, it is believed that this would benefit the hydraulic engineer and his clients of California at the expense of citizens of other classes, and even at the expense of hydraulic engineers in other parts of the country. What, for instance, would become of our navigation interests, if all regular stations were moved into the moun-

Mr. Leeds.

tains? What would become of the weather predictions, now made days ahead as a result of the early observations at regular coast stations? The Weather Bureau was created to serve the whole of the United States, and all branches of its work must be developed co-ordinately. The work of one station or locality cannot be developed with reference to the needs of that locality alone. Each section should be given the best service possible, consistent with equally good service to all other sections.

If, on page 170, line 9, there were inserted the words "by Congress and the general public", we would have a clearer statement of the truth, as the writer sees it, and it is not improbable that this was the thought in the minds of the authors.

The statement would then read as follows:

"The staff of the Weather Bureau is uniformly courteous and obliging in furnishing whatever data are available; but that these data have not a higher standard of reliability and greater practical value is due, the Committee believes, more to a lack of recognition, by Congress and the general public, of the great service that could and should be performed by the Weather Bureau, than to any reluctance on the part of those at its head to extend the scope and improve the value of its work."

As stated previously, the duties assigned by Congress to the Weather Bureau may be briefly summarized as making climatic observations and forecasting weather (particularly storms) for the benefit of agriculture, commerce, and navigation. This, broadly interpreted, would include all extensions recommended by the authors; but irrigation and hydro-electric development have proceeded so rapidly during recent years that the Weather Bureau has been unable, with the facilities provided by Congress, to keep pace with the requirements of the hydraulic engineer. Some extensions have been made, as already noted, but the attention of Congress should be directed to the need of facilities for still greater work. The Engineering Profession is probably in a better position to bring this to the attention of Congress than any other body of men. It is hoped that the present discussion will help to attain that end.

If the Weather Bureau has not sufficient funds to fulfill the duties assigned to it (as it is believed it has not), and if there are other allied duties which the Bureau should be directed and empowered to perform (as it is believed there are), then Congress should pass the necessary legislation to accomplish these ends.

The writer has had occasion, in connection with his work as a member of the Board of Engineers for Flood Control of Los Angeles County, to take up, with the present Chief of the U. S. Weather Bureau, the question of establishing additional co-operative stations in the different water-sheds in Los Angeles County. The Chief as well as

the local official and the Section Director of the Weather Bureau have expressed their willingness to co-operate to the extent of their ability. Mr. Leeds.

Another needed extension of the service of the Weather Bureau which has come to the notice of the writer is the lack of wind observations along the coast. This lack of information was brought home to him when he attempted to make a study of erosion along the California coast in recent years and the various factors affecting it, such as winds, tides, ocean currents, etc. So far as the writer knows, the only sea coast stations south of San Francisco for the observation of wind velocities and directions are at San Diego and San Pedro. The latter has been established only a short time, but will be of great service. The one at San Diego, being sheltered by Point Loma, is inadequate for showing wind velocities along the coast.

It is hoped that the Chief of the U. S. Weather Bureau will participate in the discussion of this paper and will describe fully the present work of the Bureau, its aims, its limitations, and how the Engineering Profession and the general public can assist the Bureau in making extensions of its present great service.

Fred. H. Tibbetts,* Assoc. M. Am. Soc. C. E. (by letter).—The writer is particularly pleased with the constructive criticism contained in this report, and in the subsequent discussions. There is a strong and rapidly growing sentiment favoring the participation by organized engineers in the discussion of public problems. Engineers are the chief users of the data gathered by the Weather Bureau, and hence should be particularly qualified to support it and to urge the extension by Congress of its exceedingly useful work. Mr. Tibbetts.

Among the many important matters listed by the authors as pertaining to the Weather Bureau, the writer wishes to discuss the relation of the work of that Bureau to flood control and drainage problems, particularly in the Central Valley of California.

In the Sacramento Valley many important cities and towns, including the State Capital, as well as large areas of rich farming land, lie below the flood-plain of the river, and are partly protected by incomplete flood-control and drainage works. The accurate prediction of floods is a matter of the greatest importance. Every general flood has been attended by losses running from small amounts up to sums exceeding $5 000 000. The Weather Bureau deserves the greatest credit for predicting all the important floods, and for the prompt collection of the data issued in the daily river bulletins during the flood season, as well as for the uniform courtesy and devotion of its staff. Under the present organization, and with the limited funds available, however,

* San Francisco, Cal.

Mr. Tibbetts.

it is impossible to predict gauge heights with any great degree of accuracy, or more than 1 or 2 days in advance.

The typical winter storms of California are areas of low pressure which become detached from the much larger area which remains more or less permanently in the vicinity of the Aleutian Islands. These detached areas are swept eastward over the North Pacific Ocean, by the prevailing eastward drift of the atmosphere, until they impinge on the western shore of the continent. Their progress eastward is then determined by the direction from which they have advanced, and largely by local conditions. The storms are frequently broken up, or the direction of their movement is changed, by the high mountain ranges. The location of high-pressure areas at the time the storms strike the Coast is also important. A high-pressure area over Western Canada tends to deflect a storm southward; such an area over Central or Southern California tends to hold it to the north, and make it move parallel to the Canadian boundary. In general, these storms cannot be predicted on the Pacific Coast until they have actually arrived, and as they move ordinarily at a rate of from 10 to 15 miles per hour, storm warnings are too late to be of much value. Disastrous floods in a big river system such as the Sacramento occur only as the accumulative result of a number of such storms occurring in such quick succession that heavy precipitation continues for a number of days or weeks. If, when a heavy rainfall is occurring in the Sacramento Valley, it was possible to know that there was or was not, one, two, or more other storms a day or two off the coast, it would then be possible to predict with much greater accuracy, and for a much longer period in advance, the heavy and continued precipitation which causes high and prolonged flood stages in the river. For such work, two or three, or even one, station in the ocean from 500 to 1 000 miles off the western coast, would seem to the writer to be of as much value as all the other stations in California taken together. Such stations, if established in the ocean, would be not unlike some of the lightships off the coast. Only a limited outlay, and a limited number of observers would be necessary. The stations, of course, should be equipped with wireless, so that storms moving over the ocean could be reported immediately as to intensity, direction, and rate of movement. It is probable that, if regular reports were received from all ships equipped with wireless moving in the North Pacific, many of the severe storms could be accurately foretold a number of days before they arrived. One of the authors has suggested to the writer the occasional special use of naval vessels. The lines of ocean travel from Puget Sound to the Orient appear to go far enough north to make such observations of value.

Mr. Tibbetts.

The establishment of additional Weather Bureau Stations in the higher altitudes, as recommended by the authors, would also be of much use in predicting floods. The importance of the temperature of the air, particularly in the mountains, is almost equal to that of the precipitation. During the maximum discharge of the larger rivers, all but a negligible proportion of the water comes from the higher altitudes, the proportion probably being not unlike that of the annual run-off. Very few of the Weather Bureau Stations are high enough to indicate the conditions which give high rates of run-off. The fortunate fact that the Southern Pacific crosses the mountains at comparatively high altitudes, and is giving constant and valuable support to the Weather Bureau, is of great assistance. The effect of high temperatures in producing high rates of run-off has not received from engineers the study which it deserves, and to this can be largely attributed some of the apparently erratic results of the very wet January of 1916. During the heavy storms of that month the river at no time reached a dangerous stage, probably because the temperature in the mountains was so low that the precipitation piled up in temporary storage as snow. The highest stage of the river reached thus far in 1916, accompanied by a prolonged, heavy run-off (although at no time dangerous), occurred in response to comparatively light precipitation, accompanied by unusually warm weather during the first half of February.

Colusa Basin comprises the northeast corner of the flood basin of the Sacramento River. Back of the Reclamation Districts, which include more than 120 000 acres, there is a water-shed of about 1 600 sq. miles. About 950 sq. miles of this are occupied by comparatively level valleys and plains, and the remainder, producing most of the run-off, is composed of the bare foot-hills of the Coast Range rising to elevations of from 1 500 to 2 000 ft. The flood-control works now under construction are designed primarily to handle the run-off from these hills at a maximum rate of perhaps 25 000 sec-ft. There are only four (all co-operative) stations in this drainage area, all of which are below 200 ft. elevation. Rainfall records at these stations do not necessarily give an index of the run-off. The first flood of 1916, from January 3d to 8th, aggregating about 110 000 acre-ft., was accompanied by an unusually violent storm, averaging in the valley nearly 4 in. of precipitation in less than 24 hours. This indicated satisfactorily the crest of this flood 4 or 5 days in advance. The next and larger flood occurred between January 24th and 28th, and was not accompanied by any unusual precipitation of record at the stations in the water-shed mentioned previously. Had the stations been at the elevations at which the precipitation occurred, which caused this flood, they would probably have indicated its size and time of occurrence. This is inferred from the fact that this storm is shown clearly by the

Mr. Tibbetts.

rainfall records at Kennett, which although 120 miles farther north, and on quite a different water-shed, is at an elevation of 730 ft. The writer concludes that the vertical distribution of rainfall stations is far more important, from the standpoint of flood control, than their geographical location.

The work of the Weather Bureau in collecting and making public with amazing promptness the actual conditions all along the river, is of the greatest value. By 9 A. M. it is possible to ascertain, by telephoning to the Weather Bureau, the precipitation at each station for the preceding 24 hours, the stage of the river at 7 o'clock that morning, and its rise or fall within the last 24 hours. The mail service is so prompt that these records are usually received through the mail on the day that they are issued. The writer has only one suggestion here, and that is that more frequent observations during critical periods, or, better still, the use of automatic instruments giving a continuous record of the river stage, would be of the utmost interest and value. For example, at Red Bluff, at the head of the Sacramento Valley, the river sometimes rises or falls 15 ft. or more in 24 hours. The variation is almost as great at Colusa, near the head of the heavy flood-control works. The highest gauge at Colusa is ordinarily within from 1 to 3 ft. of that at Red Bluff, and from 18 to 36 hours later. Daily readings at Red Bluff may miss the crest of the flood by a number of feet, and render it that much more difficult to predict accurately the stage of water which will be reached on the levees from Stony Creek to the Feather River.

The writer's conclusions with regard to the work of the Weather Bureau, as it affects flood control and drainage, are:

Two observation stations in the ocean would be of more value, in predicting several days in advance the occurrence of heavy storms, than all the present stations in California combined. Regular information from vessels in the ocean might be of much assistance.

A more uniform vertical distribution of the Weather Bureau Stations, particularly in the higher altitudes, is desirable.

More importance and more study should be given by engineers to the effect of temperature in the mountains on the maximum rate of run-off.

The Weather Bureau has developed amazing efficiency and promptness in the collection and publication of river bulletins, and its local officials are uniformly courteous and desirous of making their work of the utmost usefulness, but the prediction of continued storms, and prolonged high-water stages, for a sufficient length of time to be of much use in flood-control work is physically impossible under present conditions on the Pacific Coast. The timely warnings issued, however, have been of inestimable value in saving life and property.

Mr. Lippincott.

J. B. Lippincott,* M. Am. Soc. C. E. (by letter).—The writer wishes to express his appreciation of this paper concerning the work of the U. S. Weather Bureau in California. This subject is of such interest to the hydraulic engineers of the West that this Committee (consisting of Mr. Binckley and Mr. Lee), representing the Southern California Association of Members of the American Society of Civil Engineers, was appointed for the purpose of reviewing this work. The report of the Committee was presented at a regular meeting of the Association, an evening was given to its discussion, a resolution of commendation was passed, and its publication by the Society was requested.

The work of the Weather Bureau and its predecessor, the United States Signal Service, in arid America has been of great value to engineers engaged in the study of the water problems of the West. The writer is especially indebted to the Service for much useful information. It is relatively an easy matter to maintain rain gauges, and an exceedingly difficult one to maintain stream gauging stations throughout a term of years. For this reason the engineers have had to rely largely on records of the Weather Bureau and Signal Service as a basis of their study of the available water supply for irrigation and water power.

There is a constant tendency for the older bureaus of the Federal Government to maintain themselves in the City of Washington without adequate field inspection, particularly of the far western country. It is not efficient, and in fact usually is very unintelligent for one to endeavor to direct, year after year, from a desk in the Capital, field operations on the far side of a continent. This lack of proper executive has not alone characterized the Weather Bureau for a number of years past, but many other departments of the Government. This habit was broken away from vigorously first by the Geological Survey, and then by the Forest Service and the Reclamation Service.

The Engineering Profession should use its influence in obtaining, if possible by co-operation with the Weather Bureau, an expansion and improvement of its work along lines that will be of greater value to this Profession.

In California there are two great hydrographic features: First, the Pacific Ocean with 850 miles of coast line lying to the west, from which all moisture is derived; and, second, the great Sierra Nevadas and Coast Ranges of mountains that are the condensers which collect the vapor from the clouds and put it into the streams. The rainfall in the valleys of California is small, ranging from 5 to 10 in. in the Great Central Valley, and from 10 to 15 in. along the coastal plains of Southern California. The desert rainfall to the east of the Sierra is

* Los Angeles, Cal.

Mr. Lippincott.

less than 5 in. annually. The precipitation on the mountain crests is enormously greater, the increase in depth of annual rainfall being at the rate of approximately 0.6 in. for each 100 ft. rise in elevation on the western slopes of the ranges up to some unknown limit for from 6 000 to 8 000 ft. The water, both for the irrigation supply and the development of power, comes from these high mountains, yet in California there are eleven regularly established Weather Bureau stations, all in the valleys and none in the higher mountains, the one possible exception being on Mt. Tamalpais, which is a branch of the San Francisco office on a foot-hill near that city.

Apparently, the original purpose of the Weather Bureau was to be of primary aid to shipping, which is of first importance. Seven of the eleven official weather stations in California are on the sea coast, and the others are in low valleys. In California there is great variation in the rainfall within short distances. For instance, at Fresno, in the San Joaquin Valley, the yearly rainfall is about 7 in., and 30 or 40 miles away the precipitation on the Sierra Nevadas is from 50 to 60 in., yet there are three official stations in the Central Valley and none on this main range. In a horticultural State which relies largely on irrigation for its success, and is greatly interested in flood problems and inland waterways, this would appear to be a condition to correct. The same argument applies in Southern California. If it is impossible to discontinue any one of the seven coast stations, it is essential that additional regular stations be established in the high mountain drainage basins. Apparently, the cost of maintaining one of the city offices would be enough to support two in a forest reserve where a cabin could be built and the records kept in an open field.

There are some voluntary observers in these mountain regions, but they are rarely at the higher elevations, or at points where maximum precipitation occurs. These voluntary observations are usually fragmentary, and are not an adequate substitute for a regular station.

Both off the west coast of Europe and the west coast of America the warm currents or streams of the ocean have a profound effect on the climatic conditions of the regions on which they impinge. It is quite possible that there is some variation or swing in these ocean streams from year to year. Navigators talk of open and closed winters off the coast of Northern Alaska, and there have been remarkable groups of wet and dry years in California. Storms originate on and travel easterly from the Pacific Ocean across the continent. So far as the writer knows, nothing has been done, in a broad and systematic way, to study the fluctuation in temperatures of the ocean water off the Pacific Coast. There are light-houses at short intervals from San Diego to Seattle, and others farther north. It would be a simple matter, not accompanied by serious expense, for the Weather Bureau to arrange with the Light-house Service for the taking of daily tem-

peratures of the water of the ocean, with a view of determining the possible relations between changes of water temperatures and cycles of wet and dry years. There may be no such relation, but it is a physical condition that might be studied to great advantage. The fishing industry of the Pacific Coast is growing to prime importance, and some believe that the schools of fish in their migrations are influenced by the ocean temperatures and that the season's catch is thus governed. Mr. Lippincott.

It is respectfully submitted that the work of the Weather Bureau could be substantially improved, also, at the localities where regular stations are now established. It is recognized by engineers, and also by the old Signal Service, that precipitation records should be taken on the level places rather than on pinnacles or near obstructions. The U. S. Weather Bureau has published a record of the rainfall in San Francisco extending back to 1849. This is a composite of observations made successively at six different stations. In 1892 the station was moved to the roof of the Mills Building. There is available a rainfall record, made by Mr. John Pettee, extending back to 1865-66. Although it is said that the gauge observed by Mr. Pettee was moved to different parts of the city several times, it is believed that it was always established near the ground, as distinctive from the roof of a building. For the 27 years prior to 1892 the Pettee record shows a reasonably close relation to that of the Weather Bureau, being an average of 97.2% of it. Subsequent to this date (when the Weather Bureau station was moved to the roof of the eleven-story Mills Building) there was a decided difference in the ratio of the two records. For the period, 1892-93 to 1900-01, the Pettee* record was 134.4% of the Weather Bureau record. This may not be taken as conclusive of the ratio of records maintained on the roofs of high office buildings and those observed near the ground, but it certainly indicates a wide difference in the results. Other examples could be referred to.†

Again, in such a country as California, where a change of a few degrees in temperature in the spring or winter is apt to mean the loss of the fruit crop by freezing, it is not satisfactory to have temperature taken on the roofs of high buildings. Occasionally, we are confronted with reports of frozen fruit, and see some ice in the streets, but we are assured by the official records of the Weather Bureau, taken at its regular station, that the minimum temperatures have been well above freezing. The reverse conditions exist in midsummer. When attention is called to this we are told to apply some factor in correction, but few persons outside the Service know what these factors are. In the writer's opinion, these observations should be taken on the surface of the ground, as it is not satisfactory to say that corrections

* *Transactions*, Am. Soc. C. E., Vol. LXI, p. 561.

† Fanning's "Hydraulics", p. 64.

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may be applied to adjust this great mass of observed data to normal conditions.

There are a number of voluntary meteorological observers, mostly in the valleys of California, who, year after year, send in records of rainfall and temperature, whose stations are not inspected, and who are not given personal instructions and encouragement as to the method of their work. Although some of the rain gauges are in good locations, others are not, and may be misleading. A distinction should be made in the records. The writer knew of two old gentlemen who kept a rainfall record for 20 years, from 1882 to 1902, at a place known as Second Gerrote, at an elevation of 2 714 ft. Their gauge consisted of three tin cans wired together and set out in the open lot to catch the rainfall. The depth of rain was read in each can with an ordinary foot-rule at the end of each storm, and the three observations were averaged and carefully recorded in a notebook. Such interest in a scientific subject should be encouraged by a visit and personal commendation, and the furnishing of adequate instruments for observation and instruction. The writer has never known of an extended field inspection trip being made by any officer of the Weather Bureau to these voluntary meteorological stations in California, the records from many of which have been continually published for years. It is probably not the lack of interest on the part of the local observers, but rather because of the "system" that is passed down from Washington.

A great deal of the work done by the men of the Weather Bureau is highly valuable and greatly appreciated. Their prediction of frosts is of greatest importance to the fruit growers of California; their prediction of storms is essential to the mariner. These gentlemen deserve full credit for the thoroughness of their organization, which is believed to be the most extensive meteorological department in the world, yet it is suggested that this service, good as it is, could be substantially improved, from the standpoint of the engineer, without the necessity of large additional outlays.

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GEORGE S. BINCKLEY,* M. AM. SOC. C. E., and CHARLES H. LEE,* ASSOC. M. AM. SOC. C. E. (by letter).—The Committee is gratified to find that the discussion of the paper has been constructive, and has covered all sides of the subject. The Committee has been actuated by a sincere desire to get at the truth and to be of constructive service. The discussions are indicative of similar motives.

Although none of the Weather Bureau officials has contributed formally to the discussion of the paper, which is a matter of regret, yet the members of the Committee have had informal personal discussions relative thereto with the present Chief of the Weather Bureau,

* Los Angeles, Cal.

Professor C. F. Marvin, and the official in charge at Los Angeles, Dr. Ford A. Carpenter. These gentlemen, though not always in accord with the conclusions reached, assisted in many ways in amplifying the Committee's information. The members of the Committee desire to express their appreciation of the helpful suggestions and the co-operative attitude of these gentlemen toward the work. Messrs. Binckley and Lee.

The Committee understands that, at the request of the Cleveland Association of Members, the Local Forecaster, Mr. William H. Alexander, of Cleveland, Ohio, addressed that organization on the subject of its paper. The Committee has not been furnished with a copy of this address, but was kindly loaned one by Dr. Ford A. Carpenter, from his office files.

Before proceeding further, the Committee wishes to make clear three points in which the paper may have misled those who read it.

First, the title and presentation of data might indicate that the Committee's findings and suggestions applied particularly, or even solely, to the State of California. This was far from the Committee's intention, however, as it is recognized that California enjoys its share —and a very full share—of the effort and funds at the disposal of the Weather Bureau, and that the local Weather Bureau officials are doing all in their power to obtain and distribute the information desired by the engineers of the State. The conditions set forth in the paper were regarded as typical of all the Western States, and also the Central, Eastern, and Southern States. California was chosen for detailed investigation because of the greater degree of first-hand knowledge thereof possessed by the Committee. That there is equal need for greater study of rainfall in the region east of the Rocky Mountains is indicated by the strong editorial which recently appeared in one of the leading engineering journals.*

Second, the Committee does not wish its advocacy of improvement in the mountain climatological work of the Weather Bureau to be interpreted as supporting the idea that precipitation data collected at properly located mountain stations will yield all the information as to water supply and its variations needed for the design of hydraulic structures and projects. On the contrary, the Committee is in full accord with Mr. N. C. Grover's statement, that stream-flow records furnish the most reliable data for this purpose. It is unfortunately the case, however, that the establishment of stream-gauging stations must be limited to one or two points on the larger streams, on account of the first cost and the upkeep. There are many projects, however, both public and private, which depend on the supply from the tributaries of large streams, or from small streams. There are also many short stream-gauging records which do not extend over years of deficient precipitation, and, without proper corrections, are likely to be

* *Engineering News*, August 17th, 1916, p. 321.

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as misleading as precipitation records unsupported by stream-flow data. To fill these deficiencies, precipitation records judiciously used in conjunction with the available stream-flow records are of great value, and afford the only adequate information, obtainable at reasonable cost, with which to supplement insufficient stream-flow records. It is toward the filling of this gap, which every hydraulic engineer in the West at least recognizes as a serious one, that the Committee's efforts are directed.

Third, the Committee's primary idea, in formulating its suggestions for improvement of the Weather Bureau Service, was not an increase in the volume of data collected, although that might be an incidental result, but an improvement in quality, to be obtained by a more studied choice of locations at which data are observed, and more intensive study and interpretation of the data thus obtained. For instance, in the matter of precipitation data, the hydraulic engineer wants to know: First, the average annual areal precipitation for all portions of a given drainage area, and the annual variations therefrom; and second, the monthly, daily, and hourly precipitation at one or more points in the area, depending on its size. As Mr. W. S. Post well suggests, this information can be obtained for Pacific Coast mountain drainage areas by determining the shape of the isohyetose lines from a few years records at a number of well-selected stations, after which one or two base stations would provide all the information needed. (This, of course, would not be sufficient for regions where a large proportion of the annual rainfall occurs in detached local storms covering limited areas.) By working out a progressive programme for a State along these lines, the total number of stations reporting at any one time could be kept constant, although the geographical location of stations would vary. As contrasted with such a programme, there is the present more or less haphazard distribution of rainfall stations, without regard to the ultimate usefulness of the record. The Committee recognizes the fact that to plan and execute such a programme would require much intelligent study and supervision by a practical and technically trained field man. The Committee wishes to emphasize again, however, that its efforts are actuated by a desire to see generally inaugurated a progressive, practical, co-operative, and broad handling of meteorological problems, to the end that these problems may be solved. Mechanically observing data for the sole purpose of tabulation and accumulation in the archives does not solve these problems, no matter how many stations are reporting. . . .

The Committee wishes to correct an error which was inadvertently made in the paper. The number of co-operative stations in California reporting precipitation in 1913, as shown by the Annual Summary, is practically 300, instead of 192. Of these stations 8% are within

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25 miles of the coast, 43% are in interior valleys and foot-hills, 7% are in the Great Basin and Desert, and 42% (instead of 29%) are in regions of broken topography where elevations exceed 2 000 ft. Of these, approximately 30 (instead of 21 as stated), were above 4 000 ft. Hence, out of the total, 10% can be considered as mountain stations. It should also be stated that subsequent to 1913 there have been published reports of snowfall data at from 40 to 50 stations in the Sierra Nevada Mountains. These data have been expressed in snow depths, however, and not in terms of equivalent water, so that they are of little use for water supply estimates.

The individual discussions will now be taken up in detail: Mr. Grover suggests, as the reason that so few records have been obtained by the Weather Bureau in mountainous regions, the difficulty in devising self-regulating instruments. The Committee believes that, due to the rapid development which has occurred in the West during the past few years, there will be found many more reliable observers in mountain areas than might be supposed. This has been the experience of the members of the Committee. The Committee respectfully suggests that, if the local officers of the Weather Bureau were given more assistants and were allowed to become intimately familiar with the mountain portions of their districts, many good observers could be found, in areas from which records are at present lacking, by obtaining the co-operation of such local residents and by the establishment of stations in the isolated areas in charge of trained men. As the Committee has suggested, it should be entirely feasible to obtain the desired records within a reasonable cost without waiting for the development of automatically recording instruments. The Committee has already commented on Mr. Grover's reference to the possible misinterpretation of its original report, and wishes here to express its appreciation to Mr. Grover for having brought out this matter with such clearness.

Mr. Post's discussion contains several very practical suggestions which are especially valuable, as they are drawn from his experience in making rainfall and run-off observations over a large area of mountain drainage. With regard to the publication of early records, of which, as Mr. Post states, many exist in California, it has been stated to the Committee, by the Chief of the Weather Bureau, that if a sufficient volume of such data be assembled and compiled by the Local Association, and presented to the Weather Bureau, the latter will undertake their publication in bulletin form under separate cover. The Committee is still engaged in gathering data of this character, and will appreciate any which members or others may be able to furnish.

Mr. Charles T. Leeds, brings out very clearly the diversity of work which has been assigned by law to the Weather Bureau, and the

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limitations under which it is working with respect to the needs of the water supply interests of the West. The force of his statements has been brought home to the Committee by its own investigations, and it fully agrees that the proper method of obtaining desired improvements is through Congressional action; and furthermore, that, "The Engineering Profession is probably in a better position to bring this to the attention of Congress than any other body of men." Mr. Leeds has done well to mention the many lines of special research work undertaken by the Weather Bureau. With regard to the evaporation studies made by that Bureau at Salton Sea about 6 years ago, however, attention is drawn to the fact that the data thus obtained have never been made available to the public. The Committee agrees with Mr. Leeds that stations for the collection and distribution of climatic information should be maintained in the centers of population. The Committee believes, however, that fully equipped (or regular) observation stations should be at a distance from buildings and other objects and influences which, in cities, cause abnormal conditions. The Committee suggests that both requirements could be met by establishing regular stations in Government-owned buildings and on Government-owned land, at the military and lighthouse reservations, of which many exist on the Pacific Coast, for example, the Presidio at San Francisco, Point Arguello near San Luis Obispo, Fort Rosecrans at San Diego, etc. There are military reservations adjoining practically all large coast centers of population, at which communication is fully as good as in the congested business district. The maintenance of such stations would, without doubt, be less than that of the expensive office quarters usually occupied by the Weather Bureau, and, in addition, the abnormal conditions existing on top of office buildings would be eliminated, without sacrificing the requirement of easy communication.

The Committee has not been unaware of the course traveled by the typical storm centers of this continent, as suggested by Mr. Leeds, but believes that, by reason of the large diameter of these storm centers, atmospheric changes occur no later on the west slopes of the mountain ranges of the Pacific Coast than they do on the Coast itself, and that records obtained from judiciously selected mountain stations would be of just as great value in forecasting weather conditions. In this matter, however, the Committee heartily agrees with Mr. Tibbetts, that

> "Two observation stations in the ocean would be of more value, in predicting several days in advance the occurrence of heavy storms, than all the present stations in California combined."

The Committee is indebted to Mr. Tibbetts for his valuable and instructive suggestions, as they represent the experience and viewpoint

of an engineer familiar with conditions in Central and Northern California. Messrs. Binckley and Lee.

In connection with Mr. Tibbett's suggestion, it is proper to call attention to the fact that, in the course of an oral discussion of the matter at a meeting of the Southern California Association of Members of the American Society of Civil Engineers, William Mulholland, M. Am. Soc. C. E., made the suggestion that naval vessels be used for the purpose of observing and reporting by wireless weather conditions off the coast. Though it seems to the Committee that the primary objects of the naval establishment would probably interfere with regular service of this character, such information as might be given, incident to the position of the vessel, would certainly be of high value. A much broader and more comprehensive service could be obtained very readily, however, by establishing the practice of communicating, through the Government wireless stations, with all shipping within range of the station, whether it be naval, coastwise, or deep-sea. The simple request of the American Consul at a foreign port would be quite sufficient, in a vast majority of cases, to insure the prompt attention of shipmasters to wireless inquiries from our Government stations, as to their position, direction of wind, barometric pressure, temperature, etc., and, with such a custom of communication established, the reciprocal advantage to the shipmasters would be correspondingly great. The utilization of such a source of meteorological data would involve practically no expense, would call for no additional plant or labor, and, the Committee believes, would result in a very great improvement in the advance with which conditions could be forecast, and in forecasts of far greater reliability.

Mr. Lippincott has well expressed the administrative policy of the Weather Bureau with respect to the West. The visit of the present Chief of the Weather Bureau to the Pacific Coast in 1915, and the recent extension of the mountain climatological work of that Bureau in Southern California, would indicate, however, that an effort is being made by the present administration to bring about a change for the better. The Committee can agree with Mr. Lippincott that the reason so few field inspection trips are made, by local officers of the Weather Bureau, is because of the "system". Before a local officer can leave his station for official business of any kind, no matter how trivial, he must go through a long process of correspondence with the Washington office. This sometimes consumes a month, and even then the authorization is frequently curtailed, and is often denied. There is small wonder, therefore, that co-operative records are often broken or discontinued. Such stations should be visited at least once a year, and at no pre-arranged time, in order to insure accurate results and the active co-operation of the observer.

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As a result of the discussions and the Committee's further investigation of the subject, it is believed that the following general conclusions are warranted:

1.—That the law creating the United States Weather Bureau prescribes a wide range of duties, among which the gathering of precipitation records is but incidental.

2.—That there is an urgent demand among engineers, throughout the United States, for more complete precipitation data throughout mountain drainage areas, to be used in conjunction with stream-flow data.

3.—That the present fiscal regulations, organization, and administrative policy of the Weather Bureau are not adapted to the task of gathering complete precipitation data of the character desired by engineers.

4.—That Congressional action should be sought, either to change the organization of the Weather Bureau, so that the desired result can be accomplished, or else that the duty of observing all factors affecting stream flow be turned over to the Water Resources Branch of the United States Geological Survey, with the appropriation of sufficient additional funds to carry on the work efficiently.

The Committee is continuing its studies in this matter, and its conclusions and suggestions, as just stated, are not to be considered final. The question of the establishment of a Bureau of Public Works, which shall co-ordinate all engineering activities of the Federal Government, and prevent the present duplication of effort. is believed worthy of careful study. Any further suggestions from the membership of the Society bearing on these matters will be greatly appreciated by the Committee.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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Paper No. 1384

EARTH PRESSURES: A PRACTICAL COMPARISON OF THEORIES AND EXPERIMENTS.

By L. D. Cornish, M. Am. Soc. C. E.

With Discussion by Messrs. William Cain, G. M. Braune, F. N. Menefee, and L. D. Cornish.

Synopsis.

Of all structures which the engineer of to-day is required to design for conditions of static loading, there is probably none which is so irritating as the ordinary retaining wall. This, of course, is due to the lack of definite and correct data relative to the pressures exerted by the various kinds of earthy materials and to the variety of formulas which have been proposed for determining the amount and direction of the resultant earth pressure which acts on a retaining wall.

The writer has studied with a great deal of interest all the books and papers he could obtain concerning earth pressure theories, and the formulas or graphical analyses derived therefrom. Such study has been confined principally to a comparison of solutions based on the Rankine theory with those based on the sliding-wedge theory, and certain facts were developed which appear to discredit, in certain particulars, these commonly accepted theories. It is quite probable that other investigators have observed at least some of the same results, but, as the writer had never seen them discussed, he thought

the matter of sufficient importance to assemble the results of his studies in this brief paper.

Frequent reference will be made to the theories and formulas as either those of Rankine or Cain. A reference to Rankine is intended to relate, not only to the theory and formulas for the special cases as advanced by Rankine, but also to the solutions, based on his theory, covering all cases of inclination of a wall, as developed and published by Milo S. Ketchum* and M. A. Howe,† Members, Am. Soc. C. E. A reference to Cain relates to the solution by William Cain, M. Am. Soc. C. E., of the wedge theory as given in his paper‡ and in Ketchum's book.

Formulas to express the resultant earth pressure, as derived from the two theories, are similar, and in some cases identical; but the direction of the resultant force is different in all cases, except for a wall with vertical back and with the angle of surcharge equal to the angle of internal friction. These formulas, particularly those involving surcharge, are somewhat complicated, and it is impossible to grasp mentally their entire significance relative to the dimensions of a wall to resist the forces indicated by them. The writer has endeavored to show graphically the results obtained in actual wall design by the use of the different formulas and by values obtained in certain experiments, so that the points of interest may be discussed without resorting to mathematics. To accomplish this, and to avoid complications in equations, due to uneconomical distribution of masonry in the walls, all wall sections discussed will be triangular and designed as to width so that the resultant of the external forces shall cut a horizontal plane of the wall at the outer edge of the middle third.

Nomenclature.—The nomenclature used in Figs. 1 to 15, and discussion thereof is that used by Cain in his paper discussing Leygue's experiments, and, for the remainder of this paper, is the same as that used by Ketchum in his "Walls, Bins, and Grain Elevators". The differences are few and should cause no confusion, and the use

* "The Design of Walls, Bins, and Grain Elevators."

† "Retaining Walls for Earth."

‡ "Experiments on Retaining Walls and Pressures on Tunnels", *Transactions*, Am. Soc. C. E., Vol. LXXII, p. 403.

of both will facilitate quick comparison with the works of the authors mentioned.

h = vertical height of wall;

ch = " " " to center of pressure on AB, Fig. 1,

$c = \frac{1}{3}$;

ph = width of base of wall;

p = ratio of base to height;

e = weight of earth per cubic foot (Cain-Leygue);

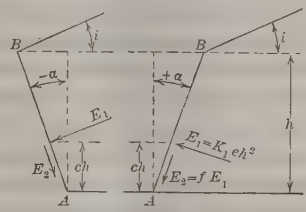


FIG. 1.

w = weight of earth per cubic foot (Ketchum);

W = total weight of earth per linear foot;

w_1 = weight of wall per cubic foot;

W_1 = total weight of wall per linear foot;

P = resultant earth pressure per linear foot of wall;

P_H and P_V = the horizontal and vertical components, respectively, of P;

$E_1 = K_1\, eh^2$ = normal component of P (Cain-Leygue);

$E_2 = E_1 \tan.\, \phi^1$ = component of P parallel to back of wall (Cain-Leygue);

k = cohesion, in pounds per square foot (Cain-Leygue);

K_1 = coefficient of E_1 (Cain-Leygue);

i = inclination of surface of earth to horizontal (Cain-Leygue);

δ = inclination of surface of earth to horizontal (Ketchum);

α = angle between back of wall and the vertical, counted as positive or negative, as in Fig. 12* (Cain-Leygue);

ϕ = angle of repose (internal friction) of the earth fill;

ϕ^1 = angle of friction of the earth filling on the back of the wall;

* Reproduced herein as Fig. 1.

θ = angle between the back of the wall and a horizontal line extending into the fill = $90 + \alpha$;

$r = \dfrac{1 - \sin.\ \phi}{1 + \sin.\ \phi}$ = the Rankine ratio of horizontal to vertical earth pressures = k, as used by Ketchum.

In discussing his solution of the sliding-wedge theory, Cain refers to various experiments, among which are those made by Leygue* and in Table 3 of his paper before this Society.† Cain gives certain results of Leygue's experiments. Leygue's experiments were made with dry sand and retaining boards, *AB*, which could be rotated about the bottom, *A*, in order to produce any desired inclination to the vertical, as shown by Fig. 1. Suitable observations were taken and results obtained, from which the values of K_1, as given in the last column of Cain's Table 3, could be computed. The reader is referred to Cain's paper† for additional details.

The values of K_1 show the relative variation in the total normal thrust against walls under different conditions of loading and inclination of the back of the walls, but convey no immediate idea of the dimensions of the walls necessary to withstand such thrusts, and, after all, the dimensions are of most interest to the practical engineer or designer.

As a preliminary to the discussion of the formulas in common use, and to provide for subsequent comparison of designs based on theory with those based on experiments, retaining walls were designed to satisfy the experimental values of K_1, as given in the last column of Cain's Table 3. In designing the walls, the unit weight of earth, *e*, was assumed at 100 lb. per cu. ft., and the weight of wall, *w*, at 140 lb. per cu. ft. The resulting designs are shown by Figs. 2 to 8, and it may be noted that the values of *e* and *h* selected are such that $eh^2 = 10\,000$, and, consequently, the normal thrust, E_1, is 10 000 times K_1, or the significant figures of E_1 of the designs and of K_1 of Cain's Table 3 are the same.

The width of the base of the walls, *ph*, was computed from the equations for *p* shown for Figs. 2 to 15, which equations are condi-

* These experiments, with discussion thereon, may be found in *Annales des Ponts et Chaussées*, November, 1885.

† "Experiments on Retaining Walls and Pressures on Tunnels", *Transactions*, Am. Soc. C. E., Vol. LXXII, p. 403.

tional on the resultant of all forces piercing the outer-third point, and the design was checked graphically as shown in the figures.

It may be noted that the required width of base steadily increases from Fig. 2 to Fig. 8. It is believed that any engineer, given these seven different cases at the same time and in a graphical form, would expect such a progressive increase, and that, in all cases, he would assert that an increase in the angle of surcharge, i, would require an increase in the width of the base. It is this point only which the foregoing discussion of Leygue's experiments is intended to emphasize for subsequent reference thereto.

In his paper, Cain showed that, in considering Leygue's experiments, the effect of cohesion should not be neglected, and, by graphical constructions and computations, he derives theoretical values of K_1, as given in the next to the last column of his Table 3 for assumed values of cohesion, k, as given in the third column of that table. It may be noted that the experimental values of K_1, as used in preparing Figs. 2 to 8, agree closely with the theoretical values, for k is equal to 1 lb. per sq. ft. Cohesion is usually neglected in practical designing, and, to show the effect of such neglect, walls, as shown by the smaller sections of Figs. 9 to 15, were designed for the theoretical values of K_1 corresponding to $k = 0$. All other assumptions were the same as those used for Figs. 2 to 8 and, therefore, the increase in section or base width is due entirely to the neglect of cohesion. The theoretical values of K_1 were derived by Cain from his theory; hence these are also the sections which result from the use of his formulas, and are marked for identification with his name. The base widths required for the same conditions were computed from the Rankine formulas, as modified by Ketchum or Howe, and the resulting sections are the larger ones marked Rankine in Figs. 9 to 15.

A study of these fourteen sections discloses three points of interest which warrant further investigation.

The first and most apparent point is the Rankine section of Fig. 12, which is smaller than the Rankine section of Fig. 11, although it is apparent by inspection that it should be larger on account of the 2:1 surcharge slope of the earth behind the wall. That it should be larger is proved by Leygue's experiments, as Fig. 5, with 2:1 surcharge, has a base 18% wider than Fig. 4, which has no surcharge. It

is also noted that the Rankine section, Fig. 13, for a 3:1 slope has a base about 1% less in width than the Rankine section of Fig. 11, which has no surcharge. These discrepancies between theory and experiment indicate that the formulas derived from the Rankine theory are erroneous, at least for certain conditions. This particular disagreement between theory on one side and reason and experiment on the other will doubtless be as surprising to many engineers as it was to the writer, and it will be discussed hereinafter in greater detail and for various conditions.

The second point of special interest is that, whereas the base of the experimental section, Fig. 4, is 30% wider than that of Fig. 3, yet the corresponding theoretical section, Fig. 11 (Cain), is 9% narrower than that of Fig. 10 (Cain). This indicates that, for walls with backs sloping toward the back-fill, the Cain and modified Rankine formulas give sections unnecessarily large. Walls of this kind are so unusual in practice that the writer, as yet, has made no further investigation of this case.

The third point is that the increase in base width of Figs. 14 and 15 (Cain) over that of Figs. 7 and 8 (Leygue) is much less than the corresponding increase for other comparable sections. This indicates that possibly the use of Cain's formulas for surcharged walls with a battered back results in sections only a little larger than for a case of no surcharge. Such is the case for walls with a vertical face and battered back, as will be shown hereinafter.

In order to put the formulas of Rankine and Cain in a comparable form, equations were deduced for p, the base width of triangular walls expressed as a percentage of the height. All the formulas for earth pressures as used are given by Ketchum,* and his nomenclature has been adopted for the following discussion in order to facilitate references.

The wall sections and equations are shown by Figs. 16 to 19 for the Rankine formulas, and by Figs. 20 to 23 for the Cain formulas. For the derivation of p, as shown, it is required that the resultant of all forces shall cut, and that the moments shall be taken about, the outer-third point of the base. The derivation of p for Figs. 16, 17, 18, and 20 is evident by inspection, as Figs. 17 and 20 require only the solving of the quadratic equation of moments.

* "The Design of Walls, Bins, and Grain Elevators."

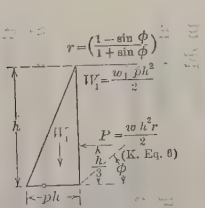


FIG. 16.

$$\frac{w_1 p^2 h^2}{6}=\frac{w h^2 r}{6}$$

whence $p=\sqrt{\frac{w}{w_1}r}$

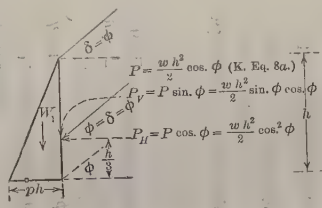


FIG. 17.

$$\frac{w_1 p^2 h^2}{6}+\frac{2\,w\,p\,h^2}{6}\sin.\phi\cos.\phi-\frac{w h^2}{6}\cos.^2\phi=0$$

whence

$$p=-\frac{w}{w_1}\sin.\phi\cos.\phi+\sqrt{\left(\frac{w}{w_1}\sin.\phi\cos.\phi\right)^2+\frac{w}{w_1}\cos.^2\phi}$$

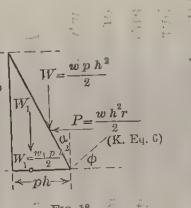


FIG. 18.

$$\frac{w p^2 h^2}{6}=\frac{w h^2 r}{6}$$

whence

$$p=\sqrt{r}$$

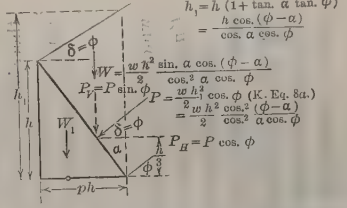


FIG. 19.

Let M = moment of a force, so that $M\,P_1$ may be read as the moment of force, P_1. Then $M\,W+M\,P_1-MP_H=0$, and, by substitution and reduction,

$$p=\sqrt{\frac{1-\sin.^2\phi}{1+\sin.^2\phi}}$$

Notes:- Figs. 16 to 19 accord with Rankine Theory.
" 20 to 23 " " Cain Theory.
Moments are taken about o, and it is required that the resultant of all forces pass through o.
The width of bases: ph, are drawn to same scale for assumption that $\phi=33°41'25''$.
(K. Eq.—) refers to the numbered equations in Ketchum's "Walls, Bins and Grain Elevators."

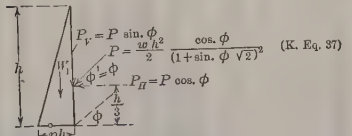


FIG. 20

$$\frac{w_1 p^2 h^2}{6}+\frac{2\,w\,p\,h^2}{6}\frac{\sin.\phi\cos.\phi}{(1+\sin.\phi\sqrt{2})^2}-\frac{w h^2}{6}\frac{\cos.^2\phi}{(1+\sin.\phi\sqrt{2})^2}$$

whence

$$p=-\frac{w}{w_1}\frac{\sin.\phi\cos.\phi}{(1+\sin.\phi\sqrt{2})^2}+\sqrt{\left(\frac{w}{w_1}\frac{\sin.\phi\cos.\phi}{(1+\sin.\phi\sqrt{2})^2}\right)^2+\frac{w}{w_1}\frac{\cos.^2\phi}{(1+\sin.\phi\sqrt{2})^2}}$$

FIG. 21.

This figure and equations are identical with those for Fig. 17.

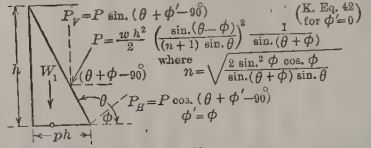


FIG. 22.

Similarly as for Fig. 19 $MP_1-MP_H=0$ whence it may be shown that $p=\sqrt{r}$ or the same as for Fig. 18

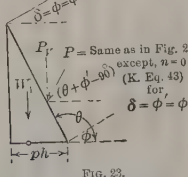


FIG. 23.

Similarly as for Fig. 22 it may be shown that

$$p=\sqrt{r}$$

For Fig. 19, the earth thrust, P, against the wall is the same as the thrust against the vertical plane, bc, of height, h_1. The height,

$$h_1 = h + h \text{ tan. } \alpha \text{ tan. } \phi$$

$$= h \frac{(\text{cos. } \phi \text{ cos. } \alpha + \text{sin. } \phi \text{ sin. } \alpha)}{\text{cos. } \phi \text{ cos. } \alpha} = \frac{h \text{ cos. } (\phi - \alpha)}{\text{cos. } \phi \text{ cos. } \alpha}.$$

By substituting this value of h_1, the equations shown on Fig. 19 are obtained, which equations are in a convenient form for practical problems.

It is somewhat easier to derive p by substituting for h_1 its value, $h\ (1 + \text{tan. } \alpha \text{ tan. } \phi)$.

The equation of moments now becomes,

$$\frac{wh^3}{6}\Big[\text{sin. } \phi \text{ cos. } \phi\ (1 + \text{tan. } \alpha \text{ tan. } \phi)^2 \text{ tan. } \alpha + (1 + \text{tan. } \alpha \text{ tan. } \phi) \text{ tan.}^2 \alpha$$

$$- \text{cos.}^2 \phi\ (1 + \text{tan. } \alpha \text{ tan. } \phi)^2\Big] = 0;$$

whence $$\text{tan.}^2 \alpha\ (1 + \text{sin.}^2 \phi) - \text{cos.}^2 \phi = 0;$$

and $$\text{tan. } \alpha = \sqrt{\frac{1 - \text{sin.}^2 \phi}{1 + \text{sin.}^2 \phi}} = p.$$

For Fig. 22, the equation of moments is,

$$P \text{ sin. } (\theta + \phi - 90) \frac{h \text{ tan. } (\theta - 90)}{3} = P \text{ cos. } (\theta + \phi - 90) \frac{h}{3};$$

whence $$\text{tan. } (\theta - 90) = \text{cot. } (\theta + \phi - 90) = p;$$

or $$\text{cot. } \theta = \text{tan. } (\theta + \phi);$$

and $$\text{cot. } \theta - \text{tan. } \theta - 2 \text{ tan. } \phi = 0;$$

but, $$\text{cot. } \theta = -p;$$

therefore, $$p^2 + 2p \text{ tan. } \phi - 1 = 0;$$

whence, $$p = -\text{tan. } \phi + \sqrt{\text{tan.}^2 \phi + 1}$$

$$= \sqrt{\frac{1 - \text{sin. } \phi}{1 + \text{sin. } \phi}} = \sqrt{r},$$

which is the same as for Rankine Fig. 18. It also may be shown that P_H and P_V (Cain) are equal in amount to P and W, as shown in Fig. 18.

For Fig. 23 it may be shown in a similar way that $p = \sqrt{r}$, but, in this case, P_H and P_V are greater in amount than for the similar forces of Fig. 22. (They are four times as great when $\phi = 30°$.) Therefore, an analysis of Fig. 23 shows that the surcharge merely produces greater compressive stresses in the wall, and the equations for p

indicate that surcharging a wall does not increase its tendency to overturn so long as the unit compression stress remains within safe limits. Such an indication is so contrary to one's preconceived ideas of the effect of surcharge that the theory from which it results can scarcely be accepted until it is substantiated by experimental proof.

It may be noted that for Figs. 18, 19, 22, and 23, the equations for p show that the width of the base is independent of the weight of either the wall or the back-fill.

The equations for Figs. 16 and 17 give identical values of p for $\frac{w}{w_1} = \frac{100}{140}$ and $\phi = 0°$, 30°, or 90°; but p for Fig. 17 is the lesser for values of ϕ between 30° and 90° and greater for values less than 30°, although the difference is not marked, as will be shown hereinafter. The narrow base resulting from the Rankine surcharge formula is due to the assumption that the resultant earth thrust is parallel to the inclined surface, as this assumption introduces a vertical component which produces a resisting moment sufficient to balance the excess overturning moment due to the surcharge. This virtually means the assumption of frictional resistance on the back of the wall, an assumption which Cain makes for all cases in his solution of the sliding-wedge theory.

The figures and equations for p on Figs. 16 to 23 show inadequately the inconsistencies of both theories when applied to various cases, but they show that neither theory gives results for surcharged walls which accord with the inference drawn from wall failures or from the experiments of Leygue, namely, that for any type of wall, if the surcharge is increased, the section must also be increased, provided the walls must satisfy the requirement that the resultant shall cut the outer-third point, or any other common point within the base.

The difference between the cases shown by Figs 16 to 23 is shown more clearly by Fig. 24, on which the p curves are plotted with reference to the rectangular co-ordinates, ϕ and p. The reference numbers on the curves refer to Figs. 16 to 23. The equations of the curves are either shown or reference is made to Figs. 16 to 23 on which they may be found, and, for all equations including the weight of the wall and the earth, it was assumed that $\frac{w}{w_1} = \frac{100}{140}$.

Curves 1 to 8 of Fig. 24 show the differences between the Rankine and Cain formulas and also between the formulas for the extreme conditions of no surcharge and maximum surcharge, but give no indication of the peculiar results obtained by the Rankine formula for Fig. 12 which is so much smaller than the corresponding wall, Fig. 11, which is not surcharged. This, however, is shown by the curves, $2a$ and $2b$, in the equations for which ϕ is a constant and δ a variable between the limits, zero and ϕ. These curves indicate that

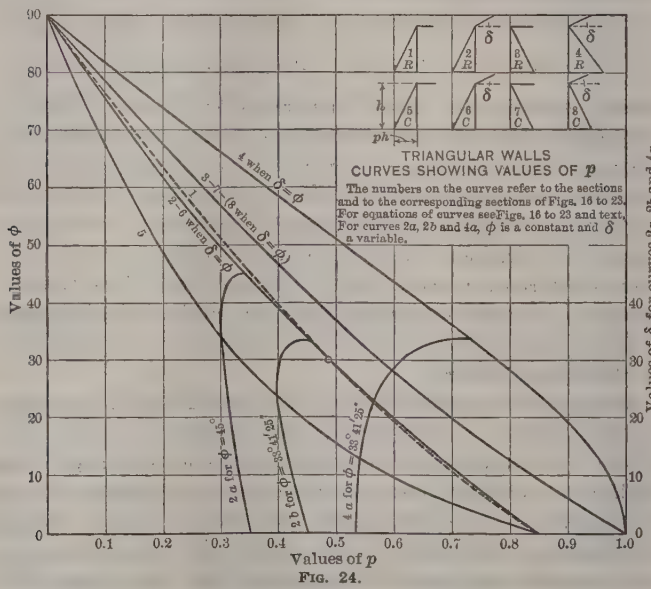

TRIANGULAR WALLS
CURVES SHOWING VALUES OF p
The numbers on the curves refer to the sections and to the corresponding sections of Figs. 16 to 23. For equations of curves see Figs. 16 to 23 and text. For curves 2a, 2b and 4a, φ is a constant and δ a variable.
Values of φ
Values of p
Values of δ for curves 2a, 2b and 4a.
4 when δ = φ
3 when δ = φ
1
5
2 a (for φ = 45°)
2 a (for φ = 35°)
4 a (for φ = 45°)
4 a (for φ = 35°)

FIG. 24.

surcharging a wall increases its stability against overturning, and that a minimum base width is required when the angle of surcharge is only a few degrees less than that of repose or of maximum surcharge. Curve $4a$ has no minimum for finite values of δ, but is similar to the others in indicating a rapid decrease in p for small decreases in δ from its maximum value of ϕ.

The equations of Curves $2a$, $2b$, and $4a$ are as follows:

Let $A = \cos.\ \delta \dfrac{\cos.\ \delta - \sqrt{\cos.^2\ \delta - \cos.^2\ \phi}}{\cos.\ \delta + \sqrt{\cos.^2\ \delta - \cos.^2\ \phi}}$, ($K.$ Equation 8), in

which ϕ is a constant; then for $2a$ and $2b$,

$$p = -\frac{w}{w_1} A \text{ sin. } \delta + \sqrt{\left(\frac{w}{w_1} A \text{ sin. } \delta\right)^2 + \frac{w}{w_1} A \text{ cos. } \delta};$$

and for Curve $4a$,

$$p = \sqrt{\frac{A \text{ cos. } \delta}{1 + A \text{ tan. } \delta \text{ sin. } \delta}}.$$

It is obvious that inferences, relative to surcharged earth pressure, to be drawn from Curves $2a$, $2b$, and $4a$, and from the similarity of Curves 1 and 2-6, are fallacious, and that, consequently, the Rankine theory—that the resultant earth thrust is parallel to the inclined surface of surcharge—is erroneous.

The writer has never been able to find or think of any good reason for assuming that the slope of surcharge should govern the slope of the resultant earth thrust for great depths, but will refrain from discussing it, as it is not the object of this paper to present a theoretical discussion, but to show, in as practical a way as the writer could devise, the results obtained from the application of the two principal existing theories.

DISCUSSION

Mr. Cain.

WILLIAM CAIN,* M. AM. SOC. C. E. (by letter).—The author refers to Ketchum's "Walls and Bins" for the writer's theories of earth pressure. The statement there is incomplete, as the writer's theory relative to the "limiting plane" is not given in that book.† The reason for introducing this conception will be made plain from the following.

In Fig. 25, let $21O$ represent the free surface of an unlimited mass of earth, subjected to no force but its own weight, $2O$ being the line of greatest declivity, making the angle, i, with the horizontal. AO represents a vertical plane perpendicular to the plane of vertical section, $A2O$. Rankine has shown (Applied Mechanics, Arts. 125, 195) that the earth thrust, E, on the vertical plane, AO, acts parallel to $2O$, or to the free surface.

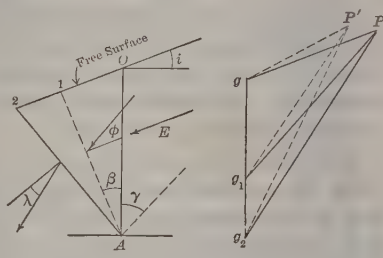


FIG. 25.

For the case supposed, there are two planes of rupture, shown by the dotted lines, which make angles, β and γ, with the vertical.‡ The thrust on the plane, $A1$, can be found by combining E, acting one-third of AO above A, with the weight of earth, $A1O$, the resultant making the angle, ϕ, with the normal to $A1$, as $A1$ is a plane of rupture. Similarly, the resultant thrust on a plane, $A2$, is found by combining E with the weight of earth, $A2O$. Suppose this resultant makes the angle, λ, with the normal to $A2$. We always have $\lambda < \phi$ (E. P., p. 38). The force diagram is shown to the right, Pg (to scale) being equal and parallel to E, gg_1 vertical and equal to the weight of $A1O$, and gg_2 vertical and equal to the weight of $A2O$. Thus, Pg_1 is the thrust on $A1$, making the angle, ϕ, with its normal, and Pg_2 is the thrust on $A2$, making the angle, λ, with its normal.

Now, in place of an unlimited mass of earth, as before, consider the earth limited by the retaining board, $A2$, which may likewise represent the inner face of a retaining wall extending to its left; and, for simplicity, take $\phi' = \phi$. It will now be shown that we cannot assume the thrust on the wall, $A2$, to make the angle, $\phi' = \phi$, with its normal and compute the thrust by the usual formulas involving wall friction.

* Chapel Hill, N. C.

† It is to be found in the writer's "Practical Designing of Retaining Walls" (D. Van Nostrand Co.), pp. 58–61, and in a more complete form in his "Earth Pressure, Retaining Walls and Bins", pp. 38–42, recently published by John Wiley and Sons. The latter book will be referred to in this discussion by the letters "E. P.".

‡ Formulas are derived in E. P., p. 102, for quickly computing these angles.

Mr. Cain.

In fact, in order that λ should equal ϕ, the graphical method (E. P., Fig. 10 and p. 38) shows that the thrust on AO must lie nearer the vertical in direction and have a less horizontal component than before. Therefore, on drawing g_2P', making the angle, ϕ, with the normal to $A2$, some line, $P'g$, lying above Pg, and having a less horizontal component than Pg, can be supposed to represent the thrust on AO. But, now, the resultant, $P'g_1$ on $A1$, lies nearer the vertical than Pg_1 and thus makes an angle greater than ϕ with the normal to $A1$. Hence, equilibrium is impossible. Therefore, λ cannot be assumed equal to ϕ, and the customary formulas cannot be used for finding the thrust on $A2$. The Rankine method is alone admissible, which consists in combining the thrust, $E = Pg$, acting parallel to the surface, with the weight of earth, $A2O = gg_2$, to find the true thrust, Pg_2, on the wall, $A\,2$. This acts at $\frac{1}{3}\,A\,2$ above A.

The plane of rupture, $A1$, is called the limiting plane. When the inner face of the wall, or retaining board, extends from A below it, the Rankine method just given must be used. If, however, the inner face of the wall or board extends from A above $A1$, then the usual (Cain) formulas apply.

For $\phi = 33° 41'$, the values of β and γ are as given in Table 1.

In Fig. 14, $i = 0°$ and $\alpha = 18° 26'$; the inner face of the wall lies above the limiting plane, as $18° 26' < 28° 10'$, and the Cain method is correctly given. In Fig. 15, however, the Cain and Rankine methods agree, and the sections should be identical, because, for $i = \phi$, $\beta = 0$,

TABLE 1.

i	β	γ
0	28° 10′	28° 10′
5°	26° 09′	30° 10′
10°	24° 08′	32° 16′
15°	21° 46′	34° 33′
20°	19° 08′	37° 11′
25°	15° 51′	40° 28′
30°	10° 59′	45° 20′
33° 41′	0°	56° 19′

and the limiting plane is vertical. The inner face of the wall lies below the limiting plane, and the Rankine method alone applies.

Similarly, the solution for Fig. 23 should be as indicated on Fig. 19, so that the remarks of the author, on pages 198-99, relative to Fig. 23, do not apply to the writer's section, which is identical with Fig. 19.

To summarize: The corrections just given refer only to Figs. 15 and 23, where the methods of Cain and Rankine are identical; so that, in Fig. 15, the Cain section should coincide with the Rankine section, and Fig. 23 should be replaced by Fig. 19.

Mr. Cain. The author has brought out the interesting fact that the Rankine section of Fig. 11, where the earth surface is horizontal, is actually larger than that of Fig. 12, where the earth surface is at the angle of repose. For very many years, the writer has pointed out the inaccuracy of the Rankine method when applied to retaining walls with vertical backs, the earth being level at top (see E. P., pages 18, 52, 277), and hopes that engineers will note the additional fact deduced by Mr. Cornish, showing the inconsistency of the Rankine method.

. The author has reached certain conclusions, in comparing Figs. 2 to 8 with the corresponding set, Figs. 9 to 15, from which the writer is constrained to differ, simply because cohesion is included in the designs, Figs. 2 to 8, but is omitted in computing the sections, Figs. 9 to 15. The variation of p for the first set then offers no criterion for judging of what its variation should be for the second set.

The experimental values of K_1, of Leygue, were found by the use of rotating boards backed by sand, the boards being 0.656 ft. (say, 8 in.) in height. The writer found that, on account of the very small height, the influence of cohesion was very marked, though only the very small value of cohesion of about 1 lb. per sq. ft., was exerted. Further, it was found, as exhibited in Table 3,* that the theoretical values of K_1, for a cohesion of about 1 lb. per sq. ft., agreed very well with the experimental values; so that sections of retaining walls, based on the experimental values of K_1, should agree very closely with the sections found by theory for earth endowed with friction and a cohesion of 1 lb. per sq. ft., for walls 8 in. in height.

For walls 10 ft. high, a larger cohesion coefficient is required than 1 lb. per sq. ft., in order to give the experimental values of K_1 pertaining to the 8-in. walls, which the author has used. By using the construction, Fig. 11, of the paper just cited, it will be found, for earth weighing 100 lb. per cu. ft., level at top and deposited behind a vertical board, that the experimental value, $K_1 = 0.090$, corresponds to about 20 lb. per sq. ft. cohesion.

Figs. 2 to 8 can then be conceived to be theoretical designs for walls 10 ft. high, backed by earth weighing 100 lb. per cu. ft., with a cohesion of 20 lb. per sq. ft., and an angle of friction, $\phi = 33° 41'$, the masonry weighing 140 lb. per cu. ft., the theoretical and experimental values of K_1 being nearly the same. Now, the theoretical sections, Figs. 9 to 15 (Cain)—Fig. 15 being corrected as above—correspond to the same earth, but without cohesion. By reference to the paper cited,† it is seen, for the experimental walls examined, of sufficient height for cohesion to be negligible, that the experimental and theoretical values (Cain) of K_1 were practically identical; so that Figs. 9 to 15 can be

* "Experiments on Retaining Walls", etc., by the writer, *Transactions*, Am. Soc. C. E., Vol. LXXII (1911), p. 421.

† *Transactions*, Am. Soc. C. E., Vol. LXXII, pp. 407–413, 426.

regarded as walls designed for experimental values of K_1 for earth without cohesion. Mr. Cain.

Thus, Figs. 2 to 8, as well as Figs. 9 to 15 (Cain) can be regarded as sections designed for experimental values of K_1, the backing, in the first set, being of coherent earth, and in the second set, of non-coherent earth. If this is true, then both sets of sections are correct, and the author is unwarranted in his assumption that the variation of p in the first set offers any criterion for judging of what the variation should be for the second set. If these sections are regarded as experimental ones, as seems highly probable from the reasoning above, then the shortcomings of the Rankine sections, Figs. 11 and 12, are made evident by the comparison.

In the sections, Figs. 9 and 10 (Cain), the earth thrust has been assumed to make the angle, ϕ, with the normal to the wall. This assumption is nearly true where α is small (say, $\alpha < 10°$), but the investigations of Résal* lead to a definite formula for this obliquity, which the writer has adopted (E. P., p. 97), and which leads to a somewhat smaller value than ϕ, for the obliquity, for $\alpha = 18° 26'$. The sections, Figs. 9 and 10 (Cain), would thus be larger and would probably approach the Ketchum-Rankine sections. Experiments on model walls, from 6 to 10 ft. high, backed by clean sand, so that the influence of cohesion is negligible, are sadly needed, in order to check, or perhaps modify, theory, particularly with respect to leaning walls. In fact, it is known from experiment, that the surface of rupture is curved and not plane, so that the theory of earth pressure against retaining walls, which assumes a plane surface of rupture, is admittedly approximate, and experiments on large models will almost certainly modify its indications.† Since there is no approximation introduced in the Rankine formula for the thrust on a vertical plane in an unlimited mass of earth, its results should agree with experiment and one would naturally expect the results pertaining to Fig. 15 (Rankine) to be verified by experiment.

Two noted authors, Boussinesq and Résal, have endeavored to complete the Rankine theory by considering the modifications necessary for those cases where the full friction of earth on wall is exerted. For such cases, both authors regard the earth thrust on the wall as making the angle, ϕ, with the normal to the wall. The case where $\phi' < \phi$ is not considered by either. Both theories are very intricate, but the results are probably more correct than those pertaining to the ordinary

* Poussée des Terres", Vol. 1.

† The modification will doubtless be more pronounced for leaning walls than for vertical or battered ones; because, when the wall leans toward the earth filling, the wedge of rupture is much smaller than for the battered walls; hence, any error due to assuming a plane surface of rupture in place of the true, curved one, should be more pronounced for the leaning walls, since, for them, the percentage of error made in the assumed wedge of rupture is greater for the leaning than for the battered walls. Further, the direction of the thrust on a leaning wall, derived from Résal's assumptions, needs experimental verification.

Mr. Cain. sliding wedge hypothesis used by the writer. The numerical values of K in Table 2 may prove interesting.*

TABLE 2.—VALUES OF K.

Earth Surface Horizontal.

α	ϕ	λ	Boussinesq. K	Résal. K	Cain. K
(1)	(2)	(3)	(4)	(5)	(6)
+ 20°	35° 00′	+ 18° 25′	.....	0.071	0.064
+ 10°	"	+ 31° 49′	.....	0.096	0.089
0°	33° 41′	+ 33° 41′	0.142	0.137	0.130
— 5°	"	"	0.159	0.156	0.152
— 10°	"	"	0.180	0.178	0.174
— 15°	"	"	0.206	0.203	0.203
— 20°	"	"	0.236	0.235	0.234
— 25°	"	"	0.274	0.273	0.272

In Table 2, α is the angle made by the inner face of the wall with the vertical, + referring to walls leaning toward the earth, and — referring to battered walls, as in Fig. 1. The angle of friction $= \phi$, the weight of earth $= w$, in pounds per cubic foot, and the thrust on the wall of height, h, is, $E = K\ wh^2$. Its component, normal to the wall, is $E_1 = K_1\ wh^2$. The thrust, E, makes the angle, λ, with and above the normal to the wall; hence $E_1 = E \cos. \lambda$ and $K_1 = K \cos. \lambda$.

The differences in the values of K given in Column 6 and in Columns 4 and 5 are greatest for the leaning walls and for a vertical wall. From values not given here, it can be seen that as the surface slope increases, the differences diminish; so that, for the earth surface at the angle of repose, for α negative, the coefficients, K, are the same by either of the three methods. Résal, alone, treats the case where α is positive.

The results of neither Boussinesq nor Résal are absolutely exact, their theories leading to differential equations that are not integrable. Boussinesq finds two limits to the thrust, and adds $\frac{9}{22}$ of their difference to the smaller to get the most probable value of the thrust. Résal assumes, as the basis of his analysis, that the stresses along any plane in a mass of earth are parallel, and increase uniformly from the surface downward. This assumption, although true for the Rankine indefinite mass, may not be true for the earth near a retaining wall. Both authors, by independent methods, reach the conclusions above relative to the "limiting plane", Fig. 25, the method of the writer, however, being much the simplest.

It will be observed, from Table 2, that the corresponding values of K do not differ very materially, so that, with a proper factor of safety,

* The numbers in Columns (4) and (5) were made out partly by interpolation from tables, with some computation. For the Boussinesq theory, see tables by Flamant in *Annales des Ponts et Chaussées*, April, 1885. Résal's tables are given in his "Poussée des Terres".

walls designed by either method would not differ in section very materially. For the design of trapezoidal walls, the writer prefers the factor of safety method, exactly as outlined in his paper* previously quoted. In this, the normal component of the thrust is alone multiplied by the factor of safety—the object being to allow, somewhat empirically, for vibration and the lubrication of the back of the wall by water from heavy rains, which may be styled the "time effect". Lubrication might be allowed for by assuming a small value for ϕ' (where the formulas involving ϕ' apply), but vibration, due to heavy trains passing at high speed, defies analysis. Mr. Cain.

The theory of earth pressure against retaining walls is a most difficult one, and eminent mathematicians have exerted their best efforts to effect an exact solution without success, though close approximate solutions have been attained—notably by Résal and Boussinesq. Therefore, any light that can be thrown on the subject is welcome, and the author's comparative sections of triangular walls are very interesting. The subject has taken on increasing interest since a Special Committee of the Society has been appointed to investigate the matter experimentally, and it is hoped that its labors may be fruitful in results.

The great difficulties in reaching correct experimental results, with the many sources of error, are now better realized than ever before, which leads one to hope that results of value may be forthcoming in the near future.

As supplementing the graphical treatment of coherent earth, as given in the writer's paper last cited, a very brief analytical solution will be given of the case of active thrust of earth with a plane upper surface, which makes the angle, i, with the horizontal. The earth, of indefinite extent, will be supposed to be subjected to no external force but its own weight, so that, by a theorem due to Rankine, the pressure on a vertical plane in its interior acts parallel to the surface.

In Fig. 26, ABC represents an infinitesimal wedge of rupture, at a vertical depth, $x = BD$, in feet, below the free surface, the wedge having a length of unity perpendicular to the plane of the paper. The lower face, AC, is supposed to make an angle, α, with the vertical, AD. The angle, α, is always acute and cannot exceed $90° - i$, when AC is parallel to the free surface. The upper face, BC, of the wedge is parallel to the free surface and AB is vertical. For brevity, let the length $AB = b$, $BC = a$, and $AC = l$. As these lengths will be supposed to be indefinitely small, the unit pressures on each face, AB, BC, AC, can be regarded as uniform.

Let w = weight of earth, in pounds per cubic foot;
ϕ = angle of friction;
c = cohesion, in pounds per square foot;

* *Transactions*, Am. Soc. C. E., Vol. LXXII, p. 433.

Mr. Cain.

p = vertical unit pressure on BC;
q = unit pressure on AB, acting parallel to the free surface;
n = normal component of unit pressure, r, on AC.

It is understood that ϕ and c have been determined by experiment,* and it must be carefully noted that, except when $c = 0$, ϕ is not the angle of repose.

The vertical prism of earth vertically over BC has a horizontal section with an area of a cos. i and a volume xa cos. i, so that vertical pressure on BC is $pa = wax$ cos. i. The weight of ABC is $\frac{1}{2} wab$ cos. i,

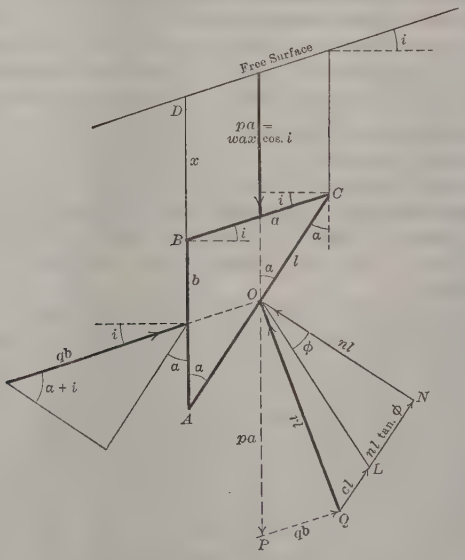


Fig. 26.

which is a second-order infinitesimal (as it involves ab); hence it can be neglected in comparison with the first-order infinitesimal, pa. The pressure on AB, acting parallel to the surface is qb, and the normal reaction on AC is nl. If motion is impending down AC, it is resisted by the full friction, nl tan. ϕ, and the cohesion, cl, both acting up the plane, AC. The resultant of the normal reaction (nl) and the tangential component ($cl + nl$ tan. ϕ) is rl, as shown on Fig. 26, where

* After the methods outlined by the writer in "Cohesion in Earth", or methods based on the same principles. *Transactions*, Am. Soc. C. E., Vol. LXXX (1916), p. 1315.

$ON = nl$, $NL = nl \tan. \phi$, $LQ = cl$, $QO = rl$, and $NOL = \phi$. The three forces, pa, qb, and rl, acting on the wedge, ABC, must be in equilibrium; hence OPQ is a closed triangle, if OP is laid off vertically (to scale) $= pa$, PQ, parallel to the surface, $= qb$, and $QO = rl$. Otherwise, if QO is replaced by its components, QN and NO, equilibrium is indicated by the closed polygon, $OPQNO$. Mr. Cain.

For equilibrium, the sum of the components of pa, qb, and the components $(cl + nl \tan. \phi)$ and nl, of rl, perpendicular and parallel to AC must be separately equal to zero.

$$p\,a \sin. \alpha + q\,b \cos. (\alpha + i) = n\,l \ldots\ldots\ldots\ldots (1)$$
$$p\,a \cos. \alpha - q\,b \sin. (\alpha + i) = n\,l \tan. \phi + c\,l \ldots\ldots (2)$$

From the triangle, $A\,B\,C$, by the law of sines,

$$\frac{b}{a} = \frac{\cos. (\alpha + i)}{\sin. \alpha}, \quad \frac{l}{a} = \frac{\cos. i}{\sin. \alpha}.$$

On dividing Equations (1) and (2) by a, and substituting the values of $\frac{b}{a}$, $\frac{l}{a}$, and clearing of fractions,

$$p \sin.^2 \alpha + q \cos.^2 (\alpha + i) = n \cos. i,$$
$$p \cos. \alpha \cos. \phi \sin. \alpha - q \sin. (\alpha + i) \cos. \phi \cos. (\alpha + i)$$
$$= n \sin. \phi \cos. i + c \cos. \phi \cos. i.$$

Substitute in the last equation the value of $n \cos. i$ from the preceding equation, transpose, and factor,

$$p \sin. \alpha\, [\cos. \phi \cos. \alpha - \sin. \phi \sin. \alpha]$$
$$- q \cos. (\alpha + i)\, [\sin. (\alpha + i) \cos. \phi + \cos. (\alpha + i) \sin. \phi]$$
$$= c \cos. \phi \cos. i;$$

or,

$$p \sin. \alpha \cos. (\alpha + \phi) - q \cos. (\alpha + i) \sin. (\alpha + i + \phi)$$
$$= c \cos. \phi \cos. i \ldots\ldots\ldots\ldots (3)$$

The true active thrust corresponds to that value of α that makes $q\,b$ or q a maximum.* For a maximum q, $\frac{d\,q}{d\,\alpha} = 0$; hence differentiating Equation (3) with respect to α, and putting $\frac{d\,q}{d\,\alpha} = 0$ (which cancels one term), we have,

$$p\,[\cos. \alpha \cos. (\alpha + \phi) - \sin. \alpha \sin. (\alpha + \phi)]$$
$$- q\,[\cos. (\alpha + i) \cos. (\alpha + i + \phi) - \sin. (\alpha + i) \sin. (\alpha + i + \phi)] = 0;$$

whence,

$$p \cos. (2\,\alpha + \phi) - q \cos. (2\,\alpha + 2\,i + \phi) = 0 \ldots\ldots (4)$$

* In Fig. 26, let PQ' represent a less thrust than the maximum, PQ, where Q' (not shown) is on PQ to the left of Q. Draw a line from Q' parallel to QN to the intersection N' with ON and lay off on it $Q'L' = QL = cl$. The new closed polygon of forces, corresponding to $OPQNO$, is $OPQ'L'N'O$. Now, however, $N'OL'$ is greater than ϕ; but ϕ being a constant, it is inconsistent with equilibrium that the angle of friction required by the new construction, $N'OL'$ should exceed ϕ; hence a less thrust than the maximum, $qb = PQ$, is inconsistent with equilibrium.

Mr. Cain. Equations (3) and (4) give the solution, which thus corresponds to the least value of q for which equilibrium is possible. This value of q is the unit conjugate thrust at B, at depth x, corresponding to the limit, as b tends indefinitely toward zero.

When $q = 0$, from Equation (4), cos. $(2\ \alpha + \phi) = 0$; therefore

$$2\ \alpha + \phi = 90°, \text{ or, } \alpha = 45° - \frac{\phi}{2}.$$

On substituting this value of α in Equation (3) and putting $q = 0$, $p = w\ x'$ cos. i, we obtain,

$$w\ x' \text{ cos. } i \text{ sin. } \left(45° - \frac{\phi}{2}\right) \text{cos. } \left(45° + \frac{\phi}{2}\right) = c \text{ sin. } (90° - \phi) \text{ cos. } i$$

$$= 2\ c \text{ sin. } \left(45° - \frac{\phi}{2}\right) \text{cos. } \left(45° - \frac{\phi}{2}\right) \text{cos. } i.$$

Therefore

$$x' = \frac{2\ c}{w} \text{ tan. } \left(45° + \frac{\phi}{2}\right) \ldots\ldots\ldots\ldots\ldots\ldots (5)$$

where x' = the vertical depth from the surface to where the conjugate thrust, $q = 0$.

In Fig. 26, the case where $q = 0$, would require $P\ Q = 0$, so that $Q\ O$, the reaction of the corresponding plane of rupture, $A\ C$, would be vertical. For $x < x'$, q remains zero and $p\ a = O\ P =$ the weight of the prism of earth vertically over $B\ C$, is entirely sustained by $n\ l$ (corresponding), combined with only a part of $(c\ l + n\ l$ tan. $\phi)$, so that the reaction, $r\ l = Q\ O$, remains vertical for any value of x from zero to x' and there is no thrust, q, on a vertical plane for the depth, x'. In fact, it can be shown generally from Equations (3) and (4) that when $0 < x < x'$, q will come out negative, or Q will lie to the left of P, indicating that a pull, $q\ b$, on $A\ B$ is necessary for equilibrium, if the full friction and cohesion on $A\ C$ is exerted. Therefore, equilibrium is possible when $q = 0$, and only a part of $(c\ l + n\ l$ tan. $\phi)$ is exerted. It follows that, for the unlimited mass of earth, subjected to no force but its own weight, there is no stress on a vertical plane in the mass, for the depth, $x = 0$ to $x = x'$.

When, from any cause, the earth tends to move away to the left of $B\ D$, a pull will be exerted on $B\ D$ for a depth, x', and its value, at depth, $x < x'$, can be found from Equations (3) and (4). This implies that the earth is capable of exerting the tension required. As clayey earths, after heavy rains, often form vertical cracks during the subsequent drying out and contraction, it seems unwise to depend on this tension, either in the case of stable slopes or retaining walls. The tension may be exerted for some time, but eventually it may and doubtless will be destroyed. In the writer's paper,* Fig. 11, page 416,

* "Experiments on Retaining Walls", *Transactions*, Am. Soc. C. E., Vol. LXXII.

refers to the case where tension is supposed to be exerted over the depth, x'; and Fig. 25, page 461, to the case where no tension is supposed to be exerted from the surface to the depth, x', given by Equation (5). Mr. Cain.

The method of solution of Equations (3) and (4) will now be indicated,* using a numerical illustration. Let $i = 30° = \phi$, $c = 100$ lb. per sq. ft., $w = 100$ lb. per cu. ft. From Equation (5), find $x' = \frac{2\,c}{w} \tan. \left(45° + \frac{\phi}{2}\right) = 2 \tan. 60° = 3.46$ ft. At this depth, as proved above, $\alpha = 45° - \frac{\phi}{2} = 30°$. Next, eliminate q between Equations (3) and (4), and substitute $w\,x \cos. i$ for p. The equation can now be solved for x, and on substituting increasing values of α, starting with $\alpha = 30°$, the values of x given in Table 3 can be found.

TABLE 3.—Values of x.

α	x, in feet.	q, in pounds per square foot.
30°	3.46	0
35°	5.26	84
40°	8.42	253
45°	15.00	646
50°	32.70	1 847
55°	124.00	8 738

The values of α give the angles made by the plane of rupture, AC, Fig. 26, for the successive vertical depths, x, from the surface, so that the curved surface of rupture, AS, Fig. 27, can be approximately drawn. It approaches indefinitely parallelism to the surface at great depths for $i = \phi$, assumed. The region, of depth x', sustains no conjugate stress. To find the values of the conjugate stress, q, at the depths, x, given above, substitute in Equation (4) the values of α corresponding and replace p by $wx \cos. i$. The values of q are quickly computed for the corresponding values of α and x given above. The conjugate stress, q, acts parallel to the surface, but it was laid off horizontally in the figure. The curve corresponding is slightly concave upward. When $i = 0$, the surface of rupture is plane; for $i > 0$, it is curved and concave upward, and the curvature increases with i. Thus, α is found to increase with the depth, and may attain the limiting value, $\alpha = 90° - i$, when AC, Fig. 26, is parallel to the surface.

* The method is very long and tedious, and not adapted to practice; hence the writer was led, in his book on "Earth Pressures", to utilize Mohr's "circular diagram of stress", not only to develop the theory of coherent earth, but also to reach numerical results with facility. See Chapter V of that treatise on the complete theory of coherent earth.

Mr. Cain. On substituting $\alpha = 90° - i$ in Equations (3) and (4), we at once find,

$$\left.\begin{aligned} p &= \frac{c \cos. \phi}{\sin. (i - \phi)}; \\ q &= \frac{c \cos. (2i - \phi)}{\sin. (i - \phi)} \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots (6)$$

Since p is always finite and positive, the case cannot occur when $i < \phi$ or $i = \phi$, but it does occur when $i > \phi$ at a certain depth, which

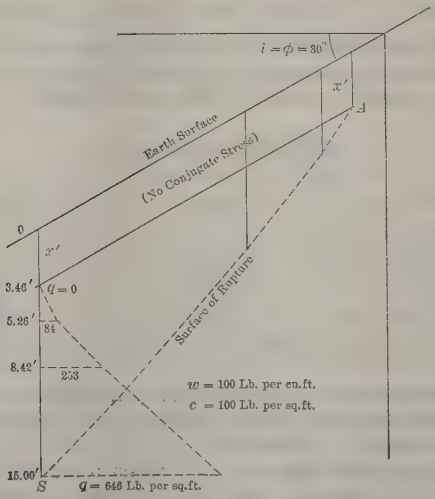


FIG. 27.

call x_0. Thus, substituting $p = wx_0 \cos. i$, in the first of Equations (6), we find the limiting depth at which the surface of rupture is parallel to the surface,

$$x_0 = \frac{c \cos. \phi}{w \cos. i \sin. (i - \phi)} \quad \ldots\ldots\ldots\ldots\ldots\ldots (7)$$

There is no equilibrium for a greater depth, since α cannot increase farther, no wedge of rupture, ABC, Fig. 26, being formed. Thus slipping is impending at $x = x_0$, and would occur for $x > x_0$, unless the earth is confined by walls, natural or otherwise, the resistance of which thus introduces external forces not contemplated in the theory of the unlimited mass, subjected to no external force but its own weight.

Mr. Cain.

If, in Fig. 26, we drop a perpendicular from B on the "free surface", and call its length h, we have $h = x_0 \cos. i$; whence, substituting in Equation (7), we derive,

$$\sin. (i - \phi) = \frac{c \cos. \phi}{w h} \quad \ldots\ldots\ldots\ldots\ldots\ldots (8)$$

From this equation, for an assumed value of h, the value of i can be found. This is the maximum inclination of the surface corresponding to the given h. After i has been thus found, $x_0 = \frac{h}{\cos. i}$ can be computed.

When i, increasing, attains the value, $45° + \frac{\phi}{2}$, the value of x_0 reduces to, $x_0 = \frac{2\,c}{w} \tan. \left(45° + \frac{\phi}{2}\right) = x'$, by Equation (5). At this depth, x' (where $q = 0$), it was found above that, $\alpha = 45° - \frac{\phi}{2}$, as should be, since $\alpha = 90° - i$ when AC, Fig. 26, is parallel to the surface. It follows, if there is to be any active thrust, that we must always have $i < 45° + \frac{\phi}{2}$.

The most interesting conclusions relative to the general case, $i > 0$, have now been given. When the earth surface slopes downward to the right from D, Fig. 26, the conjugate thrust still acting parallel to the surface, it can be shown that Equations (3) and (4) hold, on simply replacing i by $(-i)$.

It is not the intention to enter into passive thrust or resistance, but it may be stated that the solution can be effected along lines similar to those used in the case of active thrust.*

For active thrust, the most important case is that for which $i = 0$, or the free surface horizontal. From Equation (4) we obtain, at once, $\cos. (2\,\alpha + \phi) = 0$; whence, $\alpha = 45° - \frac{\phi}{2}$, for any x. It follows that the surface of rupture is plane, and that it bisects the angle between the vertical and the line making the angle, ϕ, with the horizontal.

On substituting, $\alpha = 45° - \frac{\phi}{2}$, in Equation (3), putting $w\,x$ for p, solving for q, and reducing,

$$q = \tan. \left(45° - \frac{\phi}{2}\right) \left[w\,x \tan. \left(45° - \frac{\phi}{2}\right) - 2\,c\right] \ldots. (9)$$

* In the writer's "Earth Pressures", the case of passive thrust is fully discussed by aid of Mohr's circular diagram of stress.

Mr. Cain.

This equation can be written in the form,

$$q = w \tan.^2\left(45° - \frac{\phi}{2}\right)\left[x - \frac{2\,c}{w} \tan.\left(45° + \frac{\phi}{2}\right)\right].$$

Therefore,

$$q = w \tan.^2\left(45° - \frac{\phi}{2}\right)\left[x - x'\right] \ldots\ldots\ldots\ldots (10)$$

where, Equation (5), $x' = \frac{2\,c}{w} \tan.\left(45° + \frac{\phi}{2}\right)$, is the depth at which $q = 0$, as found above for any i.

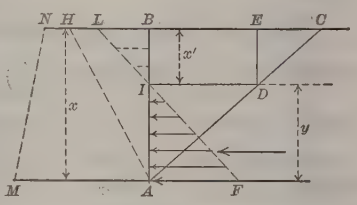


FIG. 28.

The results are as indicated in Fig. 28, where BC is the horizontal free surface, and at depth, x, q (to scale) is represented by FA. The plane of rupture is AD, and $DE = IB = x'$; so that, for the unlimited mass of earth, as shown above, there is no horizontal stress on IB or DE, the weight of EDC being supported by the vertical reaction of DC. The conjugate horizontal stress on the vertical plane, AB, is thus null on the portion, IB, and increases uniformly from I to A, so that its resultant, $\frac{1}{2}\,w\,y^2 \tan.^2\left(45° - \frac{\phi}{2}\right)$, acts at $\frac{1}{3}\,A\,I = \frac{1}{3}\,y$ above A, where $y = x - x'$.

The case is different for a limited mass of earth supported by a retaining wall, $AHNM$, which moves over slightly to the left. First, suppose AH to lie below the "limiting plane", making the angle, $45° - \frac{\phi}{2}$, with the vertical. In this case, if the earth is capable of exerting tension in the layer, $IDCB$, the stress on IB will be tensile, increasing uniformly from I to B, the unit tension at the depth, $x < x'$, being given by Equations (9) or (10), corresponding to the minus sign of q. At $x = 0$, the numerical value of this tension is $q_0 = 2\,c \tan.\left(45° - \frac{\phi}{2}\right) = L\,B$, say, and the total tensile stress on $I\,B = \frac{1}{2}\,x'\,q_0 = \frac{2\,c^2}{w}$. This stress, $\frac{2\,c^2}{w}$, acts horizontally to the right at a point on $A\,B$, $\frac{x'}{3}$ below B.

The total stress on AB is thus the difference between the resultant stresses on AI and IB, and the position of the resultant of the two can

be found by taking moments. This total stress on AB can be put in another form by integrating qdx between the limits $x = AB$ and zero. Using Equation (9), the total stress is found to be, Mr. Cain.

$$\tan.\left(45^\circ - \frac{\phi}{2}\right)\left[\frac{w\,x^2}{2}\tan.\left(45^\circ - \frac{\phi}{2}\right) - 2\,c\,x\right].$$

The tensile stress in the upper layer, of depth, x', is transmitted to DE, where its effect is to cause impending motion downward of the wedge, DCE, so that the whole of the friction and cohesion possible, is exerted on AC. In fact, if we regard ABC as the wedge of rupture, it can be independently proved that the total horizontal thrust is exactly that given by the last equation. In this case, the total thrust on the wall can be found by combining the thrust on AB with the weight of earth, AHB.

If, however, AH lies above the limiting plane, the graphical method of Fig. 11* can be applied, in order to find the stress on AH. The position of this resultant stress is uncertain. There are no tensile stresses exerted by the wall on the earth, so that the state of stress on AB is changed, particularly when AH is vertical, in which case, the stress on $AH = AB$ is wholly compressive, though exerted only over a lower portion. The state of stress in the mass, ABC, is complex, involving tension over DE, but only compression over a part of AH. As an approximation, the position of the resultant thrust on AB, for $x > x'$, can be assumed to be the same as in the first case, where AH was supposed to lie below the limiting plane, and the thrust on AH can be taken at the same or a slightly greater height.

The case will not be entered into further, because, for purposes of design of walls, it seems unwise to count on any tension in the layer, BD, as a permanent feature. Then, if AH lies below the limiting plane, the thrust on AI,

$$\frac{1}{2}\,w\,y^2\,\tan.^2\left(45^\circ - \frac{\phi}{2}\right),$$

acting $\frac{y}{3}$ above A, is combined with the weight of earth, BAH, to find the resultant thrust on the wall.

If AH lies at or slightly above the limiting plane, the same method will suffice; but, as AH approaches the vertical, this method will lead to exaggerated dimensions of the wall, and resort can be had to the graphical method of Fig. 25† in order to find the thrust on the wall.

A word may be appropriately added here as to Résal's monumental work, "Poussée des Terres, II, Théorie des Terres Cohérentes" (1910), which is the first complete treatise on coherent earth to appear. The

* "Experiments on Retaining Walls", *Transactions*, Am. Soc. C. E., Vol. LXXII, p. 416.

† *Transactions*, Am. Soc. C. E., Vol. LXXII, pp. 461, 466.

Mr. Cain. solutions of all the cases, for both active and passive thrust, are included in one general formula. In the treatment of the one case of active thrust above, the writer used a different independent variable from Résal, so that the general Equations, (3) and (4), are not the same as the corresponding ones of Résal, but are equivalent, for the case considered, and lead to the same conclusions. Résal's treatment of retaining walls, backed by coherent earth, is cumbersome, and the writer prefers a more exact method, which is in reality shorter and more convincing. This criticism, however, does not apply to Résal's general analysis of earth pressure, which will doubtless always prove as inspiring to subsequent investigators as it has to the writer.

Mr. Braune. G. M. Braune,* Assoc. M. Am. Soc. C. E. (by letter).—The writer is in sympathy with Mr. Cornish when, in closing his paper, he says:

"The writer has never been able to find or think of any good reason for assuming that the slope of surcharge should govern the slope of the resultant earth thrust for great depths."

There are two general original theories or principles for determining the earth pressure against retaining walls—namely, Coulomb's and Rankine's.

Coulomb's theory is preferable, for the main reason that the selection of the direction of the earth pressure may be assumed to fit each individual case. If there is no friction between the filling and the wall, the earth pressure would act normally to the back of the wall; if the angle of friction between wall and filling is equal to ϕ', then the earth pressure would be inclined to the normal under an angle that would approach the value of ϕ'. Whether the earth pressure is chosen normal to the back of the wall or inclined does not make very much difference in the magnitude, but it does affect considerably the overturning moment about the toe of the wall; therefore a judicious selection of the direction is an important feature in the design of walls.

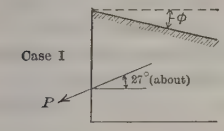
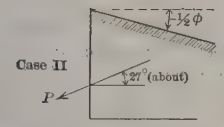
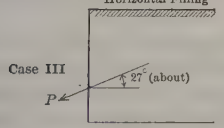
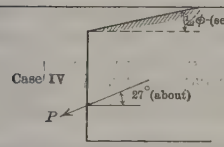
Dr. Müller-Breslau has made some experiments at the Charlottenburg Laboratory on earth pressures, and, as the writer believes that the results of these experiments are not generally known, certain parts of them will be given.

In describing these experiments, the height of wall is given as 744 mm.; the filling used was ordinary sand, weighing 1 600 kg. per cu. m. The angle of friction between the back of the wall and the sand was found to be 31° 8′, and the angle of repose 32 degrees. The average results secured on the vertical backs are shown in Table 4. The magnitude, direction, and point of application were determined for different slopes of filling behind the wall. The values, as determined from Coulomb's formula, using the angle of direction as deter-

* Cincinnati, Ohio.

mined from the experiments (27°), are given as a comparison with the experimental values. Mr. Braune.

TABLE 4.

	Magnitude of P. (1)	Direction of P to the normal. (2)	Point of application. (3)	Computed P, after Coulomb, using direction as determined from experiments. (4)
Case I 	91 kg.	About 27°.	0.31 h	89 kg. (88.6 kg.)*
Case II 	113 kg.	About 27°	0.33 h	98 kg. (106 kg.)*
Case III 	134 kg.	About 27°	0.36 h	124 kg. (122 kg.)*
Case IV 	195 kg.	About 27°	0.375 h	183 kg. (187.5 kg.)*

* The figures in parentheses in Column (4) give the value of P as computed by the writer for an angle of direction of 27°0′.

In Case 4, evidently a clerical error was made in stating that the angle of surcharge equaled ½ $\phi = 16°$, and giving a computed value of P as 183 kg. for this angle. The value, 187.5 kg., was computed by the writer, using a surcharge angle equal to ¾ $\phi = 24°$.

It will be noticed that the inclination of the filling back of the wall had practically no influence on the direction of the earth pressure, contrary to Rankine's theory. In connection with these experi-

Mr. Braune.

ments, it might be stated that no allowance was made for the frictional resistance between the material and the sides of the box, which would make some changes in the magnitude of the earth pressures, but probably would not have much effect on the direction.

In view of these experiments, Dr. Müller-Breslau recommends that the angle between the direction of earth pressure and the normal should not exceed three-quarters of the angle of repose, even with good drainage and a rough surface on the back of the wall, and provides that the resultant shall cut the base at one-quarter back from the toe.

The nature of the foundation, of course, would control the point of application of the resultant. For unyielding foundations, the resultant could safely fall outside of the middle third, provided, of course, the material is not stressed beyond a safe working limit.

It is the writer's intention to undertake some earth pressure experiments at the University of Cincinnati, commencing this fall (1916), and an endeavor will be made to eliminate the effect of frictional resistance resulting from the pressure on the side-walls.

Mr. Menefee.

F. N. Menefee,* Assoc. M. Am. Soc. C. E. (by letter).—The writer has watched, with a great deal of interest and profit, the results of Mr. Cain's plea for more light on the subject of cohesion in earth, bringing out facts bearing directly on the behavior of soil itself, as well as more information on the closely related subject of retaining walls. The following remarks are a result of the writer's attempt to choose a formula for the amount and direction of the earth pressure behind an 80-ft. retaining wall.

It is generally conceded that Rankine's formulas give conservative results, and that in most cases the modified formulas developed by other investigators are rational and well within the bounds of good mechanics. There is, however, one noticeable incongruity when the direction of the resultant thrust obtained by these formulas is compared with that obtained by simple mechanics. To illustrate this point the writer will refer to the author's diagrams and to the discussion by Messrs. Goodrich and Cain on the paper entitled "Cohesion in Earth: The Need for Comprehensive Experimentation to Determine the Coefficients of Cohesion."†

Mr. Cornish mentions the confusing differences of direction of the resultant pressure as given by different formulas, whereas, with a given coefficient of friction between the earth in question and the material in the wall, it appears to the writer that there is only one possible maximum slope to the resultant, regardless of the assumptions made as to the angle of repose or the slope of the surcharge. The writer appreciates the correctness of the formulas considering the assump-

* Jackson, Mich.

† *Transactions*, Am. Soc. C. E., Vol. LXXX, pp. 1315–1341.

Mr. Menefee.

tions, and also the part which conjugate stresses play where there is a surcharge, but believes that if the coefficient of friction of the earth against the wall can be determined, sufficient data immediately become available for finding the slope of the resultant pressure. For instance, referring to Fig. 13, the resultant pressure is given a slope equal to that of the surcharge. The magnitude, by Leygue's formula, is 4 146 lb., which, with the slope given, makes the component, parallel to the back face of the wall, 2 308 lb. In the common friction formula the tangential force necessary to move an object, divided by the normal force, is the coefficient of friction. For the case chosen, then, 2 308 ÷ 4 160 = 0.555, or there must be a coefficient of friction of 0.555 in order to give the force of 2 308 lb. parallel to the back face of the wall. In designing retaining walls, however, do we know that such a coefficient exists, between the earth and the wall, or do we even investigate it? In choosing a formula for finding the resultant pressure, it seems to the writer that we approach the problem backward and reach a conclusion which may or may not be true, for, if the coefficient of friction is not as great as 0.555, no number of formulas, however well deduced, will make it so. Suppose, for instance, that it was determined by experiment that the coefficient of friction for the earth on the wall in question was 0.35; then the maximum possible tangential force on the back of the wall would be 1 455 lb., and the slope of the resultant could not be greater than 1 455 ÷ 4 160, or about 19° 16′. Before any greater force parallel to the back of the wall could be developed, slipping would result.

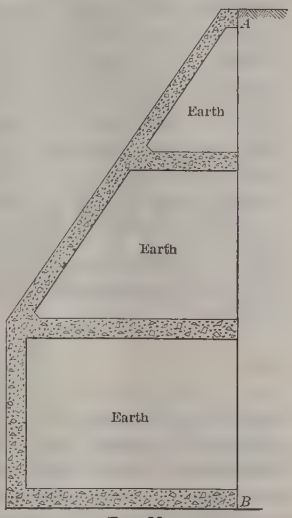


FIG. 29.

In designing some high counterforted walls of the general shape shown in section in Fig. 29, the writer found that the normal pressure of the embankment came on the earth in the cells of the wall. In this case, both friction and cohesion would act, if called on, and cause a downward force parallel to the back of the wall; but that tangential force could never be more than friction and cohesion would develop, and for that reason the writer argues that, instead of attempting to get the slope of the resultant by formula, the coefficient of friction and cohesion of the earth in question should be determined, which, when

Mr. Menefee.

multiplied by the normal force, will give the maximum tangential force that can be developed on the back of the wall.

According to the tabular values in Figs. 1 and 2 in the discussion by Messrs. Goodrich and Cain, one-fourth is not an uncommon value for the coefficient. This means that any resultant force obtained by formulas in which the slope is greater than one-fourth is incorrect, in direction at least, for its component parallel to the back of the wall would be sufficient to cause rupture along the plane, *A B*.

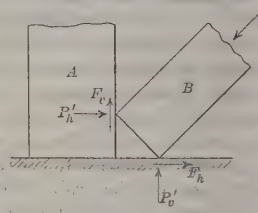


FIG. 30.

Further illustration might be made by resorting to the mechanics involved in Fig. 30. Suppose a block, *B*, is being pushed by a force, *P*, against another block, *A*, resting on the floor, as shown. The force, *P*, may be decomposed into a horizontal force, P_h, and a vertical force, P_v. Then

$$P_v = F_v + P'_v \quad \ldots\ldots(1)$$

and

$$P_h = F_h + P'_h \quad \ldots\ldots(2)$$

$F_v = P_h \times$ (coefficient of friction + cohesion), and no more. If F_v is not sufficient to equalize P_v, then P'_v must increase until Equation (1) holds. As F_v depends on the normal force and the coefficient of friction and cohesion, and F_v is equal in magnitude and opposite in sense to the active vertical component of *P*, then the writer believes that the most direct method of obtaining the vertical component of *P* is by that outlined above.

Mr. Cornish.

L. D. CORNISH,* M. AM. SOC. C. E. (by letter).—Mr. Cain corrects the writer's Sections 15 (Cain) and 23, by stating that, for these cases, where the inner face of the wall lies below his (Cain's) limiting plane, the Rankine method alone applies. Such corrections are justified by his own publications, but the sections as given are correctly computed from the equations derived from the Cain theories, by Mr. Ketchum, who, like the writer, probably was unfamiliar with the Cain theory of the "limiting plane".

Mr. Cain discusses briefly the Rankine sections of Figs. 11 and 12, and states that for years he had pointed out the inaccuracy of the Rankine method when applied to walls with vertical backs, the earth being level at the top. He extends this criticism further (E. P. p. 19) by stating his conclusion drawn from experiments "that the direction of the thrust, as given by Rankine, is never experienced in walls at the limit of stability, except where the surface slopes at the angle of repose."

* Cincinnati, Ohio.

Though the writer's Figs. 11, 12, and 13, Rankine, and Curves 1, 2, 2*a*, and 2*b*, of Fig. 24, do not in any way verify Mr. Cain's conclusions, they certainly indicate the inconsistency and unreliability of the Rankine theory and formulas when applied to walls with vertical backs. For walls with stepped or battered backs, such as Figs. 15 and 23, with large angles of surcharge, the Cain analysis, as shown by its author, is inapplicable, and the Rankine method must be used. Mr. Cornish.

Mr. Cain also states that the writer is unwarranted in his assumption that the variation of p, in Figs. 2 to 8, offers any criterion for judging of what the variation should be for Figs. 9 to 15, and, to substantiate his statement, offers certain arguments based apparently in part on the assumption that conclusions relative to walls 10 ft. high should not be based on the experimental data of Leygue for retaining boards 8 in. high. If such was his intention, it is due to a misunderstanding of the writer's criticism, as it is plainly evident that the variation in p is entirely independent of the height of the walls. The writer criticized the Cain formulas in only two particulars: one was with reference to Figs. 14 and 15 (Cain), being but slightly larger than Figs 7 and 8, which is explained by the limiting plane theory, which necessitates the use of the Rankine theory for Fig. 15. The other criticism was that the Cain formula produced a section (Fig. 11, Cain) larger than Fig. 10 (Cain), whereas a smaller section might reasonably be expected because the experimental section, Fig. 4, is smaller than the section shown by Fig. 3. It is not apparent how cohesion can account for such a variation.

The experiments of Dr. Müller-Breslau, cited by Mr. Braune, are extremely interesting, and indicate quite conclusively that the slope of the surface has little or no influence on the slope of the resultant earth pressure. It is to be hoped that Mr. Braune's proposed experiments will develop some valuable data on the vexatious question of the direction of the resultant pressure.

The writer trusts that the paper and discussions have been of some material benefit in emphasizing the necessity for systematic experimentation until sufficient data are obtained to permit the development of working formulas on which all members of the Engineering Profession can agree; and he herewith tenders his thanks to the gentlemen who have so kindly co-operated with him by engaging in the discussion.

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Paper No. 1385

A COMPLETE METHOD FOR THE CLASSIFICATION OF IRRIGABLE LANDS*

By F. H. Peters, Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. T. Kennard Thomson and G. N. Houston.

Synopsis.

This paper deals with the classification by the Dominion Government of a very large area of irrigable land developed by the Canadian Pacific Railway Company in the Province of Alberta. The reason for undertaking the work is stated. The climatic, soil, and crop conditions of the area under discussion are described, and, as governed by these, the basis of the classification is stated. Then the actual manner in which the classification of the land was made is described in detail.

The Reason for Government Classification.—The development of the Western Section irrigation block was carried out by the Canadian Pacific Railway Company. Construction was started in 1903, and the project was practically completed in 1910. The development covers a gross area of about 1 037 000 acres, of which more than 223 000 acres are irrigable. There are 17 miles of main canal, 254 miles of secondary canals, and 1 300 miles of distributaries. The total cost of construction has been $4 325 000, and it is estimated that a further expenditure

* Presented at the meeting of November 1st, 1916.

of $1 500 000 will be necessary to replace wooden structures with concrete. About 66% of the irrigable lands have been sold.

The law in force in Alberta, and under the provisions of which the project was constructed, is the Dominion statute entitled the Irrigation Act.

This Act contains a clause to the effect that, before a water license is granted to any person or corporation, a certificate must be issued, by the Chief Engineer appointed under the Act, certifying "that the works as constructed are capable of carrying and utilizing a stated quantity of water". It is necessary to certify to the carrying capacity of the works because this has a fixed ratio to the irrigable area, based on the legal duty of water, which is fixed. The Minister of the Interior, who is created by the Act the high executive officer of the Crown, decided that in the case in question the fullest intent of the quotation above should be carried out, and ruled that "utilization" should be held to mean "beneficial utilization".

This decision made it necessary to make a complete classification of all the irrigable land, and this had to be based on a rigid definition of the words "beneficial use" in terms of dollars and cents.

Determination of the Basis for Classification.—It was admittedly a difficult problem to assign a cash value to the term "beneficial use", which made necessary a balancing of the added returns of irrigation against the added cost of irrigation—both in comparison with dry farming.

The so-called "dry farming" method of agriculture was generally practiced in the Province, and the value of the returns from this method of agriculture were known. The value of returns from irrigation agriculture in the district was not known, because there had been very little actual irrigation. It was necessary to make a conservative estimate of the added returns over dry farming due to irrigation.

The added cost of irrigation was considered to be made up of four items: the additional first cost of the irrigable land over non-irrigable land; the annual charge for water; the ordinary cost of smoothing and leveling the land to allow of an even spreading of the water; and the concentrated cost of reaching, from the head-gate, certain areas of land which, being cut off by a depression, required embankment or flume construction, or, being in a depression, required drainage. The

first two items were known, and the average cost of smoothing and leveling was estimated.

The governing element in the basis of the classification, which will be described in detail later, was the item of concentrated cost.

In making an estimate of the added returns of irrigation, it was necessary, because of the lack of actual data for the district, to make a very broad study of all the essential features, and these are dealt with under the headings of "Climate", "Topography and Soil", and "Crops".

Climate.—The tract lies approximately 120 miles north of the International Boundary and 50 miles east of the foot-hills of the Rocky Mountains, and has a general slope from west to east. The elevation of the western boundary is about 3 300 ft. above sea level and that of the eastern end is about 2 900 ft.

The climatic conditions are very similar to those in Northern Montana and the higher valleys of Colorado, where irrigation has been practiced successfully for years. The average growing season between spring and fall killing frosts (32° Fahr.) is about 101 days.

Tables 1 and 2 show the temperature and precipitation for Calgary and Gleichen, which are nearly at the west and east ends of the tract, respectively. The irrigation season extends from May 1st to September 30th.

Topography and Soil.—The topography is generally rolling, with rather steep slopes which locally run up to 10 or 15 ft. in 100 ft. There are many cross-drainage lines, and, in order to avoid expensive crossings, the locators were forced to drop the elevation of the canals, thus losing command of some of the higher and better lands. The topography is somewhat smoother in the eastern than in the western portion of the tract.

The soil conditions over such a large area, of course, have a wide variation. Extreme conditions, varying from almost pure sand to very heavy clay, may be found, but, speaking generally, the soil is the usual clay loam commonly found on the prairies of the Northwest. As a whole, the area is overlaid with a surface soil containing a good proportion of humus, which proves very fertile. In general, the subsoil is a heavy clay, making it imperative to apply the irrigation water with care, in order to avoid water-logging the land. Local conditions where the soil is heavily impregnated with white alkali are found, but black alkali is rare. As will be indicated later, great care was taken

TABLE 1.—Table of Precipitation and Temperature at Calgary, Alberta.

Year.	Precipitation, in Inches.		Mean Temperature, in Degrees, Fahrenheit.	
	Total for year.	Total for irrigation season.	For year.	For irrigation season.
1885	12.91	9.32	37.05	53.6
1886	11.32	5.98	38.04	56.2
1887	13.69	9.12	33.86	54.1
1888	17.51	11.29	35.15	54.6
1889	11.59	6.41	39.54	54.7
1890	14.94	10.86	35.68	54.5
1891	10.44	8.74	37.71	55.0
1892	7.91	5.13	36.12	53.6
1893	11.05	7.17	31.76	53.9
1894	11.71	8.02	37.17	55.3
1895	15.12	10.99	36.66	53.1
1896	16.05	8.12	36.00	55.6
1897	20.58	15.02	37.10	57.8
1898	16.58	11.84	37.80	56.6
1899	26.15	21.46	34.70	53.0
1900	17.57	12.18	38.60	54.1
1901	22.31	16.47	39.20	53.3
1902	34.57	30.75	37.00	52.7
1903	22.77	19.91	37.50	52.6
1904	11.89	8.71	36.90	54.2
1905	14.12	10.02	39.00	54.6
1906	16.24	13.50	39.30	55.6
1907	14.96	11.48	36.70	52.66
1908	18.25	15.68	40.69	56.02
1909	16.03	11.98	35.97	55.78
1910	11.79	8.53	37.88	54.98
1911	19.38	15.08	35.67	52.68
1912	21.38	16.54	39.45	54.14
1913	17.08	12.85	39.94	57.08
1914	17.71	8.97	40.52	58.02
Mean for period..	16.42	12.07	37.29	54.67

TABLE 2.—Table of Precipitation and Temperature at Gleichen, Alberta.

Year.	Precipitation, in Inches.		Mean Temperature, in Degrees Fahrenheit.		Remarks.
	Total for year.	Total for irrigation season.	For year.	For irrigation season.	
1903	15.83	13.41	3	.....	
1904	10.22	7.09	37.09	55.08	
1905	11.94	9.61	37.96	55.02	
1906	17.73	13.36	38.08	55.90	
1907	.....	.....	.....	.....	
1908	.....	13.83	.....	54.90	Records not complete.
1909	20.51	17.13	34.62	55.82	
1910	.....	6.91	40.18	57.08	
1911	.....	10.88	.....	53.56	
1912	.....	6.20	.....	55.06	
1913	.....	10.16	.....	.....	
1914	9.90	5.55	.....	.....	
Mean for period....	14.36	10.38	37.59	55.30	

in the classification to exclude as non-irrigable any areas in which the soil analysis showed this great enemy to irrigation to exist in dangerous quantity.

Crops.—The leading crops in the Province are hard wheat, oats, and barley. The hardier strains of alfalfa grow excellently under irrigation, and small fields throughout the tract produce from 3 to 4 tons per acre in two or three cuttings. Experimental plot work has indicated that a heavy yield of sugar beets, with a high saccharine content, can be raised under irrigation. The irrigation tract under consideration is so young in development as yet that no good general crop reports can be obtained. In the Lethbridge District, about 100 miles to the south, however, there are about 15 000 acres under the alfalfa crop which produce about 4 tons per acre under irrigation.

For purposes of comparison, the following points are noted with reference to this tract. The general elevation is about 2 850 ft.; the soil conditions are more favorable as regards texture and absence of alkali; the annual precipitation is about 2 in. less; and the annual mean temperature is about 4½° Fahr. higher.

The Dominion Government Experimental Farm at Lethbridge has kept records of the returns from irrigated and non-irrigated crops since 1908, and Table 3 has been compiled by using these actual returns and inserting the prices which were obtained each year, f. o. b. cars, at Lethbridge. This table must be accepted as showing results from plots on the experimental farm, which are no doubt higher than those obtained by the average farmer, but it is considered that the table shows truly the relative increase due to irrigation. To be indicative of the crops commonly raised under irrigation in the district, the table should include alfalfa and timothy hay.

Basis of Classification Adopted.—The basis of classification adopted was as follows: Land shall be classified as irrigable:

(1).—If it lies at a lower elevation than the point of delivery, after allowing a reasonable grade for a farm lateral.

The point of delivery shall be deemed to be a point 3 in. below the crest of the measuring weir at or within the farm boundary. Where this measuring weir is not at or within the farm boundary, the point of delivery shall be the elevation of full supply level in the lateral supplying the field outlet, and all measuring weirs shall be built in

accordance with the plan of farm weirs filed with the Commissioner of Irrigation and dated November 1st, 1908.

The grade of farm laterals, when a factor in land classification, will be considered reasonable:

(*a*) Where the natural slope of the ground is less than 0.10 ft. in 100 ft. at a rate not less than 0.05 ft. in 100 ft.;

(*b*) Where the natural slope of the ground is greater than 0.10 ft. in 100 ft. at a rate of not less than 0.10 ft. in 100 ft.;

(*c*) Where the slope of the ground is at or near the critical slope of 0.10 ft. in 100 ft. at either of the grades mentioned in clauses (*a*) and (*b*), such as good and reasonable practice demands.

Extremely flat country may be irrigated by checks and flooding, and the grade of the farm lateral need not be considered in such cases, it being understood that the Minister of the Interior, or officers appointed by him, shall be the final judges of what constitutes "extremely flat country".

(2).—If such land can be reached by an estimated concentrated expenditure at one or more points, for embankments, flumes, etc., not in excess of $8 per acre for the land to be served and benefited through them.

(3).—If such land can be served by a second or other delivery at a cost not in excess of $8 per acre for the land to be served and benefited through them.

(4).—When lying in a depression, if it can be drained at a cost not exceeding $8 per acre, and is suitable, arable land which would be benefited by irrigation.

(5).—The prices to be made use of in estimating the cost of embankments, flumes, or other structures, shall be, unless otherwise ordered by the Minister of the Interior:

Earth fill or excavation.....12 cents per cu. yd.
Lumber in place on any structure.....$40 per 1 000 ft. b. m.
Rock rip-rap.....50 cents per superficial yard.

(6).—In all unusual or exceptional cases, which are not covered by the preceding paragraphs, the classification will be made by officers of the Department, under the direction of the Minister of the Interior, in

TABLE 3.—Records of Returns from Irrigated and Non-Irrigated Lethbridge, Alta.,

1908.			1909.			1910.			1911.		
Dry.	Irrigated.	Gain by irrigation.	Dry.	Irrigated.	Gain by irrigation.	Dry.	Irrigated.	Gain by irrigation.	Dry.	Irrigated.	Gain by irrigation.
Wheat,											
Bushels, 34	34	0	29	37	8	15	29	14	Hailed.		
$30.94	30.94	0.00	27.55	35.15	7.60	12.81	24.76	11.95			
Oats,											
Bushels, 80	88	8	56	77	21	21	68	47	Hailed.		
$28.00	30.80	2.80	18.34	25.22	6.88	5.67	18.36	12.69			
Barley,											
Bushels, 55	60	5	41	64	23	12	42	30	Hailed.		
$20.35	22.20	1.85	15.32	23.92	8.60	5.64	19.74	14.10			
Potatoes,											
Bushels, 92	235	143	159	605	446	103	521	418	356	560	204
$41.40	105.75	64.35	71.55	272.25	200.70	46.35	234.45	188.10	160.20	252.00	91.80

a fair and reasonable manner, and will be based on the beneficial use of water.

Much attention was given to the question of the maximum slope permissible for land to be classified as irrigable, but this was not included in the official basis. The engineer in charge, in approving the classification, was guided by the rule that generally a maximum slope of 10 ft. in 100 ft. was permissible, and where very small areas only were concerned a maximum slope of 12 ft. in 100 ft.

As indicated under the heading, "Determination of the Basis for Classification", the classification was based on a cost of $8 per acre,

GATED CROPS AT THE DOMINION GOVERNMENT EXPERIMENTAL FARM CANADA, FROM 1908 TO 1914.

1912.			1913.			1914.			Averages.		
Dry.	Irrigated.	Gain by Irrigation.	Dry.	Irrigated.	Gain by Irrigation.	Dry.	Irrigated.	Gain by Irrigation.	Dry.	Irrigated.	Gain by Irrigation.
RED FIFE."											
31	63	32	27	45	18	20	67	47	26	46	20
26.62	54.10	27.48	20.05	33.41	13.36	14.78	49.50	34.72	22.13	37.98	15.85
" BANNER."											
77	145	68	73	115	42	49	113	64	59	101	42
26.08	49.12	23.04	19.71	31.05	11.34	14.64	33.76	19.12	18.74	31.38	12.64
" CLAUDE."											
29	81	52	40	94	54	30	97	67	34.5	73.0	38.5
10.37	28.96	18.59	12.40	29.14	16.74	11.10	35.89	24.79	12.53	26.64	14.11
" IRISH COBBLER."											
296	501	205	229	528	299	400	495	95	234	492	258
133.20	225.45	92.25	103.05	237.60	134.55	180.00	222.75	42.75	105.10	221.46	116.36

but as this is a very crucial point, many details are given in order to explain just what the figure means.

Under the conditions of its land sale contract and water agreement, the Company delivers the water to at least one point on each parcel of land, usually of 160 acres, and the cost figure was applicable only to the added expenditure necessary for the farmer to distribute the water over the land. The figure was not a general one applied to the total irrigable area in each parcel, and including the ordinary costs of leveling and smoothing the land, but was the concentrated cost of reaching certain areas of land which, being cut off by a depression,

required embankment or flume construction, or, being in a depression, required drainage.

The following study, which was made by the writer in examining the cost data before the basis of classification was adopted, will make the point quite clear.

Referring to Fig. 1, the area, *A-D-E,* can be easily irrigated without any high expenditures for flumes, embankments, etc., and the $8 figure does not recognize any expenditure necessary for this area. The area, *H-J-K-L,* is separated from the field outlet by a wide depression, and it required a costly raised ditch and flume to carry water from *B* to *C*. The basis of classification means that if the cost of the raised ditch and flume from *B* to *C* is less than a concentrated ex-

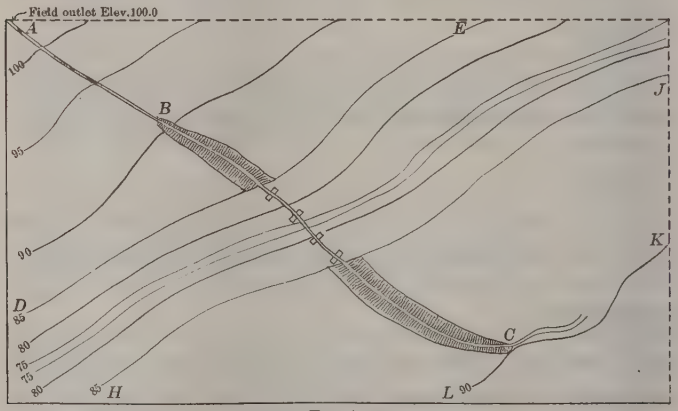


FIG. 1.

penditure of $8 per acre on the area, *H-J-K-L,* then *H-J-K-L* shall be classed as irrigable. Again, the $8 figure does not recognize the ordinary costs of preparation necessary for the irrigation of *H-J-K-L* after the water has been conducted to *C*. Now, in truth, it would require a certain expenditure per acre in smoothing the land, building field laterals, distributaries, etc., to irrigate *A-D-E,* and similarly a certain expenditure per acre would be required to irrigate *H-J-K-L* after water had been conducted to *C*.

In considering how much may be profitably spent in preparing land for irrigation, it is usual to compute it at so much per acre for the whole parcel of land, this figure including all flumes, embankments,

ditches, etc., and also the smoothing of the land to allow of proper irrigation.

The writer will now endeavor to show what the concentrated cost of $8 per acre means for different cases, and for the average case, couched in the usual terms, as explained in the foregoing paragraph.

If it is assumed that the average piece of land will require an expenditure of $4 per acre for smoothing, ditches, etc. (there being no clearing of any kind necessary in this district), and if this is combined with the concentrated cost of $8 per acre, it is seen immediately that the actual total cost of preparing any land which requires the "concentrated" expenditure is really $8 plus $4, or $12 per acre. It must be realized that in every parcel of land, considered as a unit, there will be a varying proportion which can be prepared for irrigation with an expenditure of only $4 per acre, and of other land which will require in addition to this the "concentrated cost" expenditure of $8 per acre.

TABLE 4.—Cost per Acre for Structures and Preparation of Land for Irrigation.

No. of acres of good land.	Cost per acre.	Total cost.	Percentage of land requiring concentrated expenditure.	Number of acres.	Cost per acre.	Total cost.	Grand total, in acres.	Grand total cost.	Average cost per acre.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
100	$4	$400	0	0	$12	0	100	$400	$4.00
90	4	360	10	10	12	$120	100	480	4.80
80	4	320	20	20	12	240	100	560	5.60
70	4	280	30	30	12	360	100	640	6.40
60	4	240	40	40	12	480	100	720	7.20
50	4	200	50	50	12	600	100	800	8.00
							600	$3 600	$36.00
					Averages.....		100	$600	$6.00

Note.—Average cost per acre for structure and preparation of land for irrigation, say, $6.

It is not claimed that Table 4 is absolutely accurate in any way, but it shows that, for the average piece of land, the cost distributed over the whole irrigable area will be about $6 per acre, and that for the piece where 50% requires the concentrated expenditure (which

must be considered a high percentage) the cost per acre in a similar way will only be about $8.

Since the classification has been completed, this study has been checked by actual results, and it has been found that the average total cost per irrigable acre, including the $4 charge for smoothing and leveling, has been $5.70.

On 8%	of the farms the cost was	less than	$5.	
On 60%	" " " " " "	between	$5	and $6.
On 22%	" " " " " "	"	6	" 7.
On 8½%	" " " " " "	"	7	" 9.
On 1%	" " " " " "	"	9	" 10.
On ½%	" " " " " "	"	10	" 12.

Surveys and Plans Required.—It was realized that the basis of the very complete classification anticipated must be very accurate contour plans showing the detailed topography of all the area to be classified. The Canadian Pacific Railway Company was required to furnish the necessary plans, and as it had made topographical surveys covering only a small portion of the total area, it was necessary to make new surveys of the remainder.

The specifications for these plans were as follows:

1.—The scale of the plans shall be 400 ft. to 1 in.

2.—On every plan shall be shown 5-ft. contours in heavy lines, and wherever the slope is less than 2 ft. in 100 ft., 1-ft. contour lines shall be shown in thin lines.

3.—All "ring" contours in hollows or on summits shall be located on the ground by at least four points.

4.—All sloughs or other areas covered by water shall be located by at least four points.

5.—The tracings submitted in duplicate to the Commissioner of Irrigation shall be made from the original plane-table plans made in the field.

6.—The location and elevation of every field outlet shall be shown plainly on each plan. The elevation of the field outlet shall be a point 3 in. below the crest of the measuring weir, if located at or within the farm boundary; where this measuring weir is not at or within the farm boundary, the elevation of the field outlet shall be the elevation of full supply level in the lateral supplying such outlet.

7.—The area and location of such land as is commanded by the ditches, but which is plainly rendered non-irrigable owing to alkali or any other cause, shall be shown on the plans.

8.—No contour lines need be shown above the falling contour which passes through the elevation of the field outlet and falls from it at the following rates:

(*a*) Where the natural slope of the ground is less than 0.10 ft. in 100 ft., at a rate not less than 0.05 ft. in 100 ft.;

(*b*) Where the natural slope of the ground is greater than 0.10 ft. in 100 ft. at a rate not less than 0.10 ft. in 100 ft.;

(*c*) Where the natural slope of the ground is at or near the critical slope of 0.10 ft. in 100 ft. at either of the rates mentioned in (*a*) and (*b*), such as good and reasonable practice demands.

9.—Every precaution shall be taken to make the plans as accurate as possible within the limit of a reasonable expenditure of money. The maximum error in the plans area delineated by the critical contours shall not exceed 2½% in any quarter-section of land.

10.—The exact location of any of the Company's distributaries or secondary canals which run through any parcel of land, shall be the outside lines which determine the land that is made non-tillable by the ditch construction or other works.

11.—All distinct drainage lines or channels shall be traced on the ground and shown on the plans.

12.—All plans submitted should be to a standard, over-all size, for the purpose of filing, and should not exceed 18 by 18 in. This will permit one section to be shown on each plan at a scale of 400 ft. to 1 in., and one quarter-section at a scale of 200 ft. to 1 in.

13.—It is understood that, under Secondary Canal *C*, about 90 000 acres of land have already been contoured to a scale of 1 000 ft. to 1 in. For this particular land, Paragraph 5 of these specifications shall be inoperative.

14.—It is understood that, under Secondary Canals *A*, *B*, and *C*, about 100 000 acres of land have already been surveyed to a scale of 200 ft. to 1 in. These surveys do not show contours, but show a classification of land, based particularly on soil conditions. These plans with descriptive notes will be accepted by the Commissioner of

Irrigation, who will reserve the right to call for such additional information on any particular plan as he may require for the purpose of the re-classification.

With reference to Clause 13 of the Specifications, this meant in practice that the plans submitted were to the general scale of 400 ft. to 1 in., but were enlarged from the original field plans (at 1 000 ft. to 1 in.) which had previously been prepared by the Company.

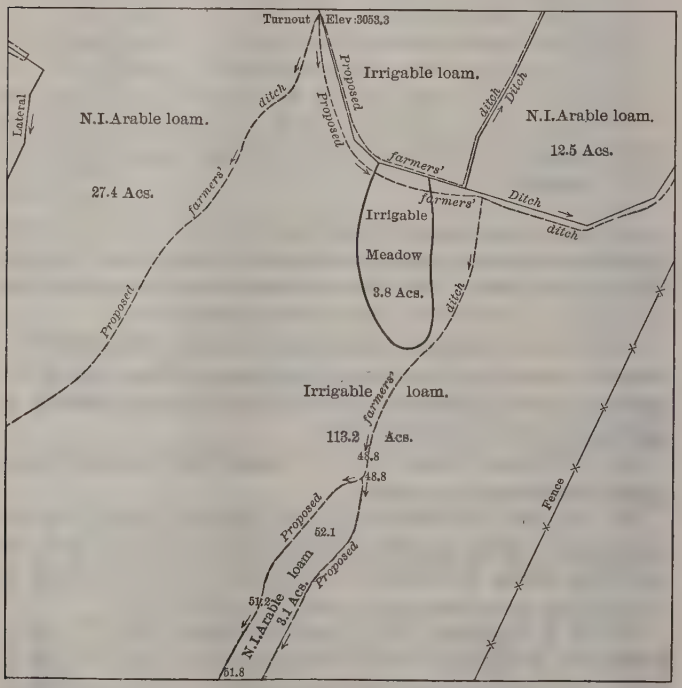


FIG. 2.

The left half of Plate III shows a typical plan as submitted by the Company, and Fig. 2 shows a typical plan as submitted under the special Clause 14 of the Specifications.

The Company has constructed all the canals and ditches which deliver the water to the field outlet for each parcel of land. The usual

parcel to which a delivery is made is 160 acres, this being a legal quarter-section, the parcel usually dealt with in selling. In making the classification it was necessary to know the exact parcel of land that was to be served from any field outlet.

In the case of land already sold by the Company, agreements filed with the Department showed the parcels owned and included under each water agreement, and in these cases such areas were considered as one. In other special cases, usually concerning fractional areas, the Company's plans submitted showed by a conventional sign the special parcels that were to be sold together, and were so dealt with as regards parceling in the classification.

The cost of preparing the plans, under the general clauses of the Specifications, was 23 cents per acre, this being based on the total area, irrigable and non-irrigable, covered by the contour plans.

Procedure of Classification.—The procedure of carrying out the classification was as follows:

1.—Submission of contour plans by the Company;
2.—Preliminary office classification of plans;
3.—Field classification of plans, including soil analysis;
4.—Final office classification of plans;
5.—Final approval of plans by the engineer in charge.

Preliminary Office Classification.—The Canadian Pacific Railway Company operates at Calgary a lithographic print process, and, in addition to the original tracings, supplies an extra plan on paper.

The paper copy of the plan was then classified in the office by projecting the necessary farmers' ditches from each farm delivery shown and delineating irrigable and non-irrigable areas in accordance with the basis of classification decided upon.

The purpose of the office classification was to delineate the irrigable and non-irrigable areas which were dependent on topography, that is, cost of field irrigating ditches, cost of drainage ditches, excessive slopes, etc. In addition, there were excluded all areas reserved by the Company as right of way for main canals, and the areas rendered non-tillable by the smaller company laterals which ran through any farm and for which easement had been reserved in the land contract sale to the farmer.

The results of this office classification were scheduled on a "Land Classification Detail Sheet", Fig. 3, this sample referring to the right half of Plate III.

The cost data for ditches, embankments, flumes, etc., used in the office classification were taken from the standard design sheets which are shown as Figs. 4, 5, 6, and 7, which were worked up for office use from the basis of classification determined upon.

LAND CLASSIFICATION DETAIL SHEET
WESTERN SECTION C.P.R.

S	T	R	OFFICE CLASSIFICATION: CLASS. BY		CHECKED BY		APPROVED BY
8	23	25	A. A. S.	2/15	C. d. K.	3/21	R. C. S. OFFICE ENG.

Parcel — N.W.¼

A	86.1 Ac. Irrigable
B	33.9 " N.I. Cost above limit.
C	15.8 " " " Above Delivery.
D	8.84 " " " Slough.
E	5.1 " " " Slopes.
F	0.7 " " " "
	9.27 " " " Right of Way.
	0.29 " " " Road Right of Way

Estimate

50 Sta's	@ $1.14 per Sta.	$57.00
18 "	@ 1.25 " "	22.50
24 "	@ 1.18 " "	28.32
		$107.82

TOTALS: Irr. 86.1 N.I. 64.34 Rt.-W. 9.56 Includes 0.29 Ac. Road Diversion
Estimated Cost per acre of Ditches and Structures to Irrigate Land $1.25

N.E.¼

A	104.2 Ac Irrigable.
B	31.8 " " "
C	10.3 " N I Slopes.
D	1.8 " " " Above Delivery.
	11.9 " " " Right of Way.

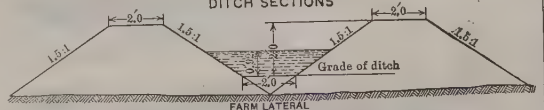
Estimate

24 Sta's	@ $1.14 per Sta.	$27.36
52	@ 1.14 " "	59.28
12	@ 1.20 " "	14.40
		$101.04

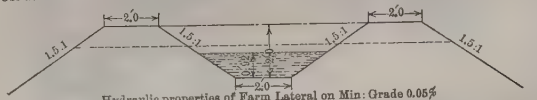
TOTALS: Irr. 136.0 N.I. 12.1 Rt.-W. 11.9
Estimated Cost per acre of Ditches and Structures to Irrigate Land $0.74

FIG. 3.

Field Classification.—The paper copies of the plans, together with the land classification detail sheet, were then sent out to the field, where, map in hand, the classification was carefully examined on the ground by the engineer in charge of the work. This engineer was supplied with a motor car for quickness of transport, and was assisted by three engineers, each accompanied by an instrumentman, a rodman, and a teamster with a democrat. The field work was most carefully examined, and, whenever any undesirable areas were found,

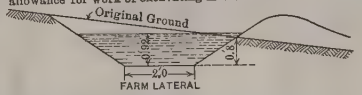


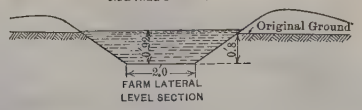
Use double fill when water level is from 0.3 to 1.7 above ground level.
Use fill as below when water level is more than 1.7 above ground, if less expensive than flume.



Hydraulic properties of Farm Lateral on Min: Grade 0.05%
$n = 0.025$ $s = 0.0005$ $Q = 2.46$ cu. ft. per sec.

Fill to be constructed to the dotted line and the ditch excavated to the section shown.
Cost data computed for fill to height of dot and dash line (top of Bank) to make allowance for work of excavating ditch.





COST OF FARM LATERAL

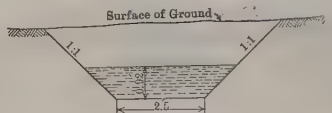
Cross Slope %	Cu. yds. per Sta. of 100′	Cost Dol'rs. per Sta.
0	9.48	1.14
1	9.84	1.18
2	10.20	1.22
3	10.91	1.27
4	11.00	1.32
5	11.40	1.37
6	11.80	1.42
7	12.25	1.47
8	12.75	1.53
9	13.20	1.58
10	13.70	1.64
11	14.22	1.71
12	14.73	1.76
13	15.30	1.83
14	15.83	1.90
15	16.40	1.97
17½	17.95	2.15
20	19.70	2.36
For Laterals up to 2.5% Max. Grade		

COST OF DOUBLE FILL

Height Ground above Sta.	Cu. yds. per Sta. +15% Shrinkage	Cost per 100′ @12¢ per cu. yd.
0.0	29.8	3.58
0.5	54.3	6.52
1.0	85.4	10.20
1.5	122.4	14.70
2.0	210.4	25.20
2.5	277.0	30.35
3.0	306.0	37.46
3.5	358.6	43.05
4.0	415.0	49.80
4.5	472.8	56.70
5.0	526.0	64.30
5.5	601.0	72.12
6.0	670.0	80.40
6.5	742.0	89.05
7.0	818.0	97.86
7.5	894.0	107.33
8.0	977.0	117.25
8.5	1061.0	127.36

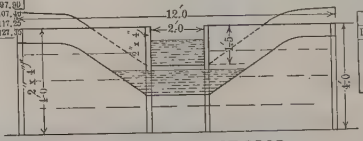
COST OF FARM LATERALS ON A GRADE BETWEEN 2.5% & 5.5%

Grade %	Dollars per 100 ft.	Number of Drops
2.5	1.14	0
3.0	5.42	1
4.0	9.70	2
5.0	13.98	3
For Ditches exceeding 5.5% Grade use a Chute		



THROUGH CUT

Depth of Grade	Cu. yds. per Station	Cost per Sta. @12¢ per cu. yd.
1.0	13.0	1.55
1.5	22.2	2.65
2.0	33.3	4.00
2.5	46.3	5.55
3.0	61.1	7.35
3.5	77.7	9.35
4.0	96.2	11.55
4.5	116.5	14.00
5.0	139.0	16.70
5.5	163.0	19.75
6.0	189.0	22.70
6.5	217.0	26.05
7.0	247.0	29.65

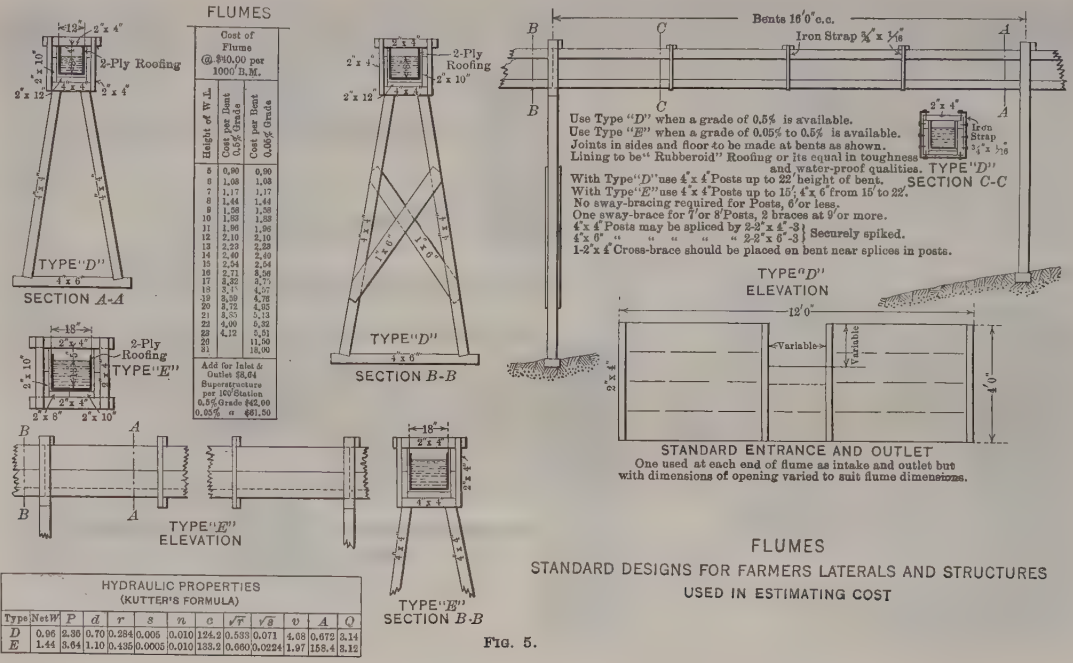


BILL OF MATERIAL

Pcs	Size	Lgth.	F.B.M.
1	2″x4″	10′	11
4	2″x12″	12′	96
			107.

DITCH SECTIONS
STANDARD DESIGNS FOR FARMERS LATERALS & STRUCTURES
USED IN ESTIMATING COST

FIG. 4.


FLUMES
12"
2"x 4"
2-Ply Roofing
2"x 10"
2"x 12"
2"x 4"
TYPE "D"
SECTION A-A
18"
2-Ply Roofing
TYPE "E"
2"x 4"
2"x 10"
2"x 8"
B
A
TYPE "E"
ELEVATION
2-Ply Roofing
TYPE "D"
SECTION B-B
TYPE "E"
SECTION B-B
Bents 16'0" c.c.
Iron Strap ¾" x ⅛"
Use Type "D" when a grade of 0.5% is available.
Use Type "E" when a grade of 0.05% to 0.5% is available.
Joints in sides and floor to be made at bents as shown.
Lining to be "Rubberoid" Roofing or its equal in toughness and water-proof qualities.
With Type "D" use 4"x 4" Posts up to 22' height of bent.
With Type "E" use 4"x 4" Posts up to 15'; 4"x 6" from 15' to 22'.
No sway-bracing required for Posts, 6' or less.
One sway-brace for 7' or 8' Posts, 2 braces at 9' or more.
4"x 4" Posts may be spliced by 2-2"x 4"-3' } Securely spiked.
4"x 6" " " " " " 2-2"x 6"-3' }
1-2"x 4" Cross-brace should be placed on bent near splices in posts.
TYPE "D"
SECTION C-C
TYPE "D"
ELEVATION
12'0"
Variable
Variable
4'0"
STANDARD ENTRANCE AND OUTLET
One used at each end of flume as intake and outlet but with dimensions of opening varied to suit flume dimensions.

Height of W. L.	Cost per Bent 0.5% Grade	Cost per Bent 0.05% Grade
5	0.90	0.90
6	1.08	1.08
7	1.17	1.17
8	1.44	1.44
9	1.58	1.58
10	1.83	1.83
11	1.96	1.96
12	2.10	2.10
13	2.23	2.23
14	2.40	2.40
15	2.54	2.54
16	2.71	3.56
17	3.32	3.75
18	3.51	4.57
19	3.39	4.78
20	3.72	4.95
21	3.85	5.13
22	4.00	5.32
23	4.12	5.51
20		11.50
21		12.00

Cost of Flume @ $40.00 per 1000 B.M.

Add for Inlet & Outlet $3.64
Superstructure per 100' Station
0.5% Grade $42.00
0.05% " $61.50

HYDRAULIC PROPERTIES
(KUTTER'S FORMULA)

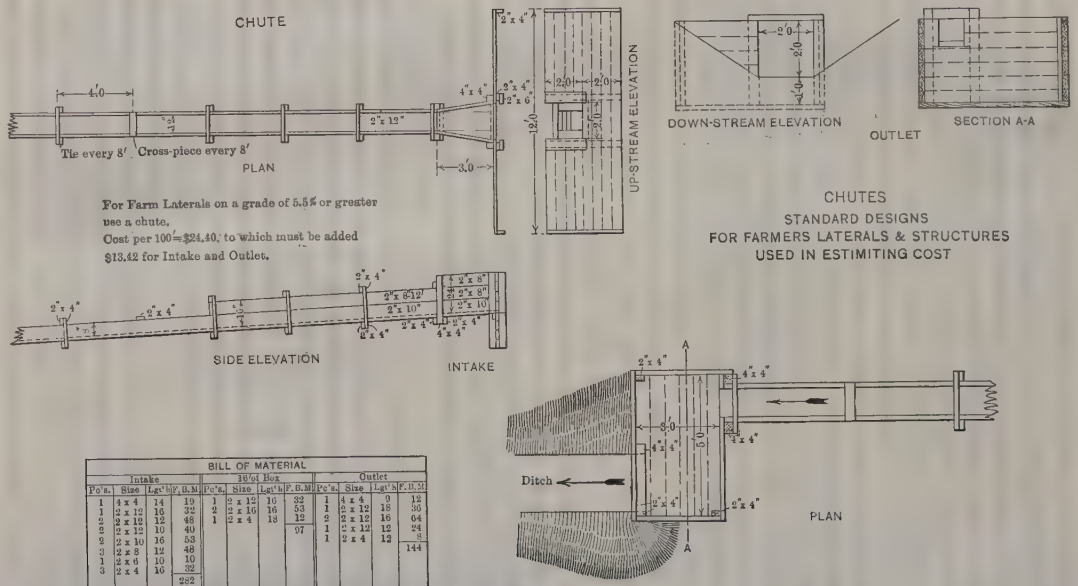
Type	Net W	P	d	r	s	n	c	$\sqrt{r}$	$\sqrt{s}$	v	A	Q
D	0.96	2.36	0.70	0.284	0.005	0.010	124.2	0.533	0.071	4.08	0.672	2.14
E	1.44	3.64	1.10	0.435	0.0005	0.010	133.2	0.660	0.0224	1.97	158.4	3.12

FLUMES

STANDARD DESIGNS FOR FARMERS LATERALS AND STRUCTURES

USED IN ESTIMATING COST

Fig. 5.



BILL OF MATERIAL

Intake				16"x16" Box				Outlet			
Pc's.	Size	Lg'th	F. B.M.	Pc's.	Size	Lg'th	F. B.M.	Pc's.	Size	Lg'th	F. B.M.
1	4 x 4	14	19	1	2 x 12	16	32	1	4 x 4	9	12
1	2 x 12	16	32	2	2 x 16	16	53	1	2 x 12	18	36
2	2 x 12	12	48	1	2 x 4	13	12	2	2 x 12	16	64
2	2 x 12	10	40				97	1	2 x 12	12	24
2	2 x 10	16	53					1	2 x 4	12	8
2	2 x 8	12	48								144
1	2 x 6	10	10								
3	2 x 4	16	32								
			282								

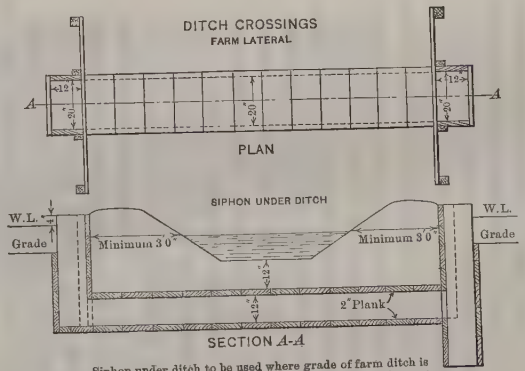
FIG. 6.

such as alkali conditions, land not fit for cultivation, or any other conditions not shown by the contour plans, these areas were delineated by the level-compass-stadia method, and, where greater detail was necessary, by the plane-table. In many cases, also, of critical cost, an actual profile was run in the field for irrigation or drainage ditches, embankments, or flumes.

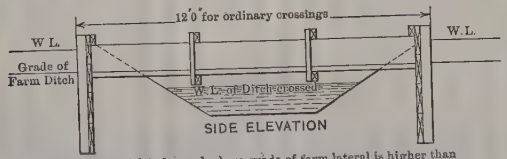
One of the most difficult features was the determination of unsuitable soil conditions, and the greatest care was exercised in excluding alkalied lands or lands likely to be alkalied by irrigation where the cost limit would not allow of efficient drainage. In connection with this work many soil samples were taken with the soil auger or post-hole digger, in order to determine the physical structure of the surface soil and the subsoil, or the presence of alkali in dangerous quantity. The quantity of alkali was determined by submitting soil samples to complete chemical analysis.

It was fully realized that the field classification was the most critical operation in the general procedure, and that, following the careful topographical surveys and the office classification, the net irrigable areas capable of beneficial irrigation could only be determined by a careful inspection in the field by an experienced man who had a full knowledge of practical agriculture under irrigation.

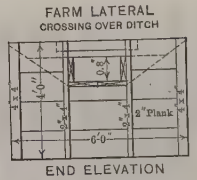
Final Office Classification.—The paper plans and classification sheets which had been used in the field were returned to the office with copious pencil notes and, where field traverses had been made or profiles run, these were plotted on separate sheets in the field and attached to the plans. One of the tracing linen plans was then worked up in the office, and included all the corrections to the office classification made in the field. The scheme adopted was to outline on the tracing the irrigable areas, the non-irrigable areas, and the right of way reserved by the Company. By the use of different colors, in accordance with an adopted schedule, the reason that each area was rendered non-irrigable was shown. Each plan showed a whole section of land, and at the bottom there was filled in a schedule showing for each quarter-section the total irrigable land, the total non-irrigable land, and the right of way reserved by the Company. The land classification detail sheets, showing the summation of the irrigable and non-irrigable areas, were also corrected. Each separate detail was referenced to the finished



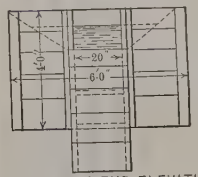
Siphon under ditch to be used where grade of farm ditch is too low to clear W.L. of ditch to be crossed. Where dimensions are not shown vary lengths to suit conditions. Allowance in estimate of cost is made to cover cost of tar between all joints of wood.



Crossing over ditch to be used where grade of farm lateral is higher than W.L. of ditch to be crossed. Give box as much fall as circumstances permit. If necessary to drop grade after crossing, make box long enough on the lower side to clear the bank of the ditch crossed.



Estimate cost of 12 ft. Crossing at $10. Add $0.35 per foot for each additional foot in length up to 16 ft.



Estimate cost of Siphon under small ditch at $25. Add $0.50 per foot for each additional foot in length up to 16 ft.

DITCH CROSSINGS
STANDARD DESIGNS FOR FARMERS LATERALS & STRUCTURES
USED IN ESTIMATING COST

FIG. 7.

plan by the use of the letters, *A, B, C,* etc., which same letters were used to denote the segregated areas on the plan.

The right half of Plate III shows the final classification plan worked up from the plans shown on the left half of that plate (different conventional edgings being used instead of colors). The land classification sheet referring to the plan shown on the right half of Plate III is also shown.

With the special plans submitted under Clause 14 of the Specifications, the result obtained was practically the same, but the procedure was necessarily somewhat different. These plans showed the result of a classification in the field by the Company's officials, who had worked under instructions which were practically the same as the basis of classification adopted. No contour lines being shown on these plans, no preliminary office classification was possible, and all the checking had to be done in the field, which necessitated considerable additional work in determining elevation and delineating areas by the field assistants. The final classification plan was prepared in the same manner as described above.

Final Approval of Plans and Classification.—The final approval was made by the engineer in charge after checking the plans to see that the field notes submitted had been properly interpreted in the office in making up the final copy. The cost of making the classification based on the total area inspected was 16.2 cents per acre, of which 7 cents is chargeable to office work. Based on the irrigable acreage only, the cost was 32.9 cents per acre with 14.1 cents chargeable to office work. These costs included making duplicates of all the classification maps for special purposes of record.

Personnel.—The work was carried out under the direct charge of G. N. Houston, M. Am. Soc. C. E., and Mr. R. C. Spitzer acted as Office Engineer.

DISCUSSION

Mr. Thomson.

T. KENNARD THOMSON,* M. AM. SOC. C. E. (by letter).—This clean-cut paper is of especial interest to the writer because he was on the survey for the first railroad, the Canadian Pacific, to be built in that section of the country, 33 years ago.

At that time nobody ever expected to see this road pay for itself, as it was built as a political necessity to keep British Columbia and the great Prairie Provinces in the Dominion.

An old farmer described the conditions then as follows: "Forty degrees below zero, 50 miles from water, 500 miles from fuel, God bless our home."

The paper would be even more interesting, if made a little more general by stating, for instance, the quantity of water required for irrigation, and whether the supply is continuous or intermittent during the summer. As the water comes from the mountains, the writer assumes that the supply would be cut off by the ice in winter. For instance, even the North Saskatchewan, at Saskatoon, a mighty river in summer, is practically dry in winter.

It would also be interesting to know what the good and poor lands sell for, before and after irrigation.

The alkali lands referred to remind the writer that he heard only last week why his hair had turned white when quite young. It was discovered, not very long ago, that all young men in the Canadian Northwest have become white-haired because of the alkali in the water and also because of the quantity of ammonia used for purifying the water.

The author states that the climatic conditions are the same as in Northern Montana, but as the range of temperature is more interesting than the mean, why not state what they are?

Is it not true that the depth of the rich top soil is 15 ft. or more in vast areas of the Canadian Northwest? It may be more modest not to extol the richness of the soil, etc., but, at the same time, one would like to know the facts. The price for excavation and fill, 12 cents (as stated by the author), seems to be remarkably low, especially as the quantities of material moved in any one spot are so small.

The sections of the cuts and excavations are so small, and the wooden superstructures are so light, that one wonders if they do not require very considerable repair every spring.

The Canadian engineers, instead of using blue or black prints on paper, make black prints directly on cloth, and these can be placed in a pocket or bag as readily as a handkerchief; and the writer is

* New York City.

Mr. Thomson.

surprised that such a great improvement has not already been generally adopted by American engineers.

Table 3, showing the increased value of land each year due to irrigation—an increase from 0 for wheat to $200 an acre for potatoes—is exceedingly interesting. In New York State, frost in a single night will sometimes destroy all the crops; and then a month or more of good weather will follow with no frosts. Is this the case in Alberta?

Some day it will be possible to generate by electricity sufficient heat for a night or two to afford sufficient protection from such frosts; and it will be possible even to go further and, by artificial heat and light, produce two crops where only one grows now.

Mr. Houston.

G. N. Houston,* M. Am. Soc. C. E. (by letter).—Irrigation development comes under one of two classes: that affecting areas where irrigation is a necessity, and that affecting areas where it is only desirable. Irrigation is a necessity where, other conditions being favorable, the precipitation is insufficient to produce a profitable crop. It is desirable where the precipitation, other conditions being favorable, is insufficient to produce the maximum profitable yield of the particular crops adapted to the region.

The possibilities under the first class have been about exhausted, except as the area is extended by conserving the water and rendering it more efficient in crop production. Under the second class, there is a great deal yet to be accomplished.

It has not been considered necessary to make any extensive study of the climatic conditions in connection with irrigation development in the United States, because the projects have been nearly all confined to the arid and semi-arid districts, where the desirability of irrigation had been demonstrated in a small way by the individual farmer, or by the small community of settlers. It was assumed that like benefits would accrue, in this district, wherever water could be applied to arable land, and the engineer has been concerned principally with transporting and applying this water.

In connection with projects of the second class, however, that is, where irrigation is not a necessity but is simply desirable for increasing the yield of an already profitable crop, the development is reaching out into the more or less humid regions, and the question of desirability is becoming more and more complicated. In these cases a study of the climatic conditions is necessary, especially with regard to the precipitation, in order to determine whether the probable additional profits due to irrigation would be sufficient to justify the cost of the system.

Monthly averages of the precipitation as published do not furnish the necessary data for this study. The daily distribution of the rainfall,

* Denver, Colo.

Mr. Houston.

or, in other words, the occurrence of drought periods during the cropping season, must be considered, as this is the chief factor affecting the yield of the crop. In connection with the work of which the paper treats, the writer had occasion to make such a study of the climatic conditions of several points in Southern Alberta and Northern Montana, a summary of which is given in Table 5. A graph was made showing the daily rainfall during 11 years for each of the locations, and the drought periods were picked out. Column 1 shows the length of each period; Column 2 shows the total precipitation which occurred during each drought; Column 3 shows the number of days during the drought on which rain fell; and Column 4 shows the maximum rainfall on any one day during the drought.

The effect which a period of light rainfall may have on a crop depends on a number of factors, such as the rate of evaporation, as affected by humidity and wind velocity, the kind of crop and the stage of its growth, the general texture of the soil, and whether or not it has had any preparation to receive and conserve the moisture. These matters must be given due weight in determining the desirability of irrigation, especially on those tracts where the climatic conditions are nearly humid.

In the development of irrigation projects, too little attention has been paid to the character of the soil of the land proposed to be put under water. This, however, is simply one phase of the general tendency of the promoter, the investor, and in many cases the engineer, to disregard the conditions under which the investment is to be returned. Large expenditures have been made in order to deliver water to land the character of which, except as regards surface appearance, has been practically unknown.

Some of this land is impregnated so heavily with alkaline salts that it cannot produce sufficient under irrigation to return interest on the investment. Where the alkali shows on the surface in large white patches, there is little danger of the land being included in an irrigation scheme without provision being made for its reclamation. The subsoil, however, may be very heavily impregnated and there may be very slight indications, or none at all, on the surface, until after the project is in operation, when the salts are brought to the surface by the direct application of water or seepage from a near-by ditch. The result is that the investor suddenly realizes that he will be obliged to build an expensive drainage system, to wash out the alkali, or cut out this land entirely from the project. In the latter case, in order that the investment shall be returned, the remaining land must be burdened with an additional tax, in proportion to the part excluded, which possibly it is not able to stand.

No irrigation project should be built until a careful study of the soil conditions has been made, not only in regard to the topography

Mr. Houston.

Group No. 164		
Sec.	T.	R.
7	23	26
Cavendish Dist.		

SOIL INVESTIGATION FIELD NOTES

June 11, 1914

(a) Apparatus used in collecting. post-hole digger no 6

(b) Approx. distance from Stream, Lake, or other water: about ½ mile S. of slough and ¼ mile E. of ditch

(c) Approx. elevation above or below water; 20' below ditch and 10' above slough

(d) Land—~~In Crop Summerfallow~~—Unbroken ~~Formerly cropped but idle.~~

(e) Present Crop None ~~Exceptionally good, Good, Average, Poor.~~

(f) If unbroken, character of native vegetation; prairie grass.

(g) ~~Irrigated~~ Unirrigated

(h) If irrigated, the date of last application of water

(i) Date of last rainfall on the area; june 9th, 1914.

(j) Are conditions where sample is taken average for the ¼ section? yes, for west half of land below ditch

(k) Does white alkali appear on the surface? yes, slight indications.

(l) Has it affected the vegetation? yes; a few small bare spots.

(m) Remarks; with thin grass around the edges otherwise, a good crop of prairie grass on this area.

Sample	Depth: from	to	Moisture	Color.	Description
A 164	0.0	0.4	M.	dark gray.	clay and loam.
B 164	0.4	2.0	D.	gray.	clay loam.
C 164	2.0	3.5	D. &	gray-brown	clay loam.
D 164	3.5	6.0	W. &	yellow.	sand.
E ---	--	--	---	------	------
F ---	--	--	---	------	------

(n) Depth of the surface soil; 0.4 ft.

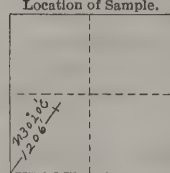
(o) Depths to which roots penetrate; 5.0 ft

(p) Depth to the water table; 5.5 ft

Remarks: hole fell in at the bottom, apparently quick sand.

Collected by:	Approved:
W. E. G. H.	G.W.H.
Asst. Field Eng.	Eng. in Charge

Location of Sample.



S. W. ¼ Section 7-23-26

FIG. 8.

TABLE 5.—Summary of the Studies of Drought Conditions in Southern Alberta and Northern Montana.

Year.	Gleichen, Alberta.				Medicine Hat, Alberta.				Lethbridge, Alberta.				Boseman, Mont.				Chinook, Mont.			
	(1) Length of period, in days.	(2) Total precipitation in period.	(3) Number of days on which rain fell.	(4) Maximum for one day.	(1) Length of period, in days.	(2) Total precipitation in period.	(3) Number of days on which rain fell.	(4) Maximum for one day.	(1) Length of period, in days.	(2) Total precipitation in period.	(3) Number of days on which rain fell.	(4) Maximum for one day.	(1) Length of period, in days.	(2) Total precipitation in period.	(3) Number of days on which rain fell.	(4) Maximum for one day.	(1) Length of period, in days.	(2) Total precipitation in period.	(3) Number of days on which rain fell.	(4) Maximum for one day.
1913	27	0.00	0	0.00	27	0.17	4	0.10	31	0.53	6	0.29	22	0.33	8	0.17	25	0.41	2	0.25
	23	0.02	1	0.02	21	0.23	2	0.22	19	0.27	4	0.20	21	0.59	4	0.25	17	0.18	1	0.18
	17	0.33	2	0.25									17	0.27	3	0.11				
1912	18	0.17	3	0.10	26	0.39	7	0.10	20	0.34	6	0.16	24	0.68	7	0.19	20	0.40	2	0.38
	25	0.34	2	0.26	17	0.17	3	0.13	23	0.07	2	0.06	23	0.64	6	0.20	39	0.48	3	0.22
	17	0.20	2	0.15	19	0.30	3	0.15									17	0.23	2	0.19
	20	0.26	3	0.10	21	0.33	6	0.17												
1911	24	0.00	0	0.00	20	0.30	8	0.10	18	0.13	3	0.06	22	0.85	5	0.41	20	0.12	1	0.12
	17	0.00	0	0.00					24	0.48	8	0.14	24	0.28	3	0.15				
1910	31	0.55	2	0.30	63	0.86	11	0.16	41	0.45	8	0.16	69	1.02	13	0.25	31	0.52	7	0.12
	64	0.00	0	0.00	31	0.53	5	0.22	39	0.08	2	0.06					37	0.08	3	0.03
1909	27	0.56	4	0.44	25	0.49	2	0.39	43	0.80	11	0.22	26	0.71	8	0.19	14	0.03	1	0.03
									19	0.15	4	0.05	14	0.12	2	0.09	27	0.50	2	0.38
1908	36	0.55	4	0.25	25	0.24	4	0.07	49	0.54	5	0.19	57	1.05	12	0.25	19	0.34	3	0.19
					32	0.44	7	0.17									43	0.19	2	0.11
					24	0.40	5	0.14									18	0.15	3	0.08
1907	19	0.14	1	0.14	19	0.05	1	0.05	21	0.43	7	0.03	17	0.56	6	0.30	19	0.07	1	0.07
	21	0.32	2	0.18	17	0.20	2	0.15									28	0.20	4	0.10
					20	0.23	5	0.10												
					24	0.30	3	0.14												
1906	19	0.30	3	0.13	13	0.00	0	0.00	31	0.61	4	0.21	38	0.50	6	0.23	43	0.09	1	0.09
	28	0.39	1	0.39	49	0.40	6	0.10	16	0.13	1	0.13								
1905	20	0.23	3	0.10	17	0.34	4	0.12	21	0.36	6	0.25	46	0.975	10	0.30	18	0.22	1	0.22
	19	0.32	3	0.22	20	0.14	2	0.08	25	0.24	5	0.08					14	0.00	0	0.00
					32	0.64	7	0.16												
1904	21	0.25	2	0.20	19	0.38	4	0.14	46	0.42	4	0.19	20	0.48	7	0.12	18	0.10	1	0.10
	45	0.39	4	0.13	43	0.84	7	0.20					17	0.06	2	0.05	21	0.08	1	0.08
					22	0.46	2	0.36					31	0.95	8	0.38				
1903	14	0.25	1	0.25	65	1.06	12	0.35	16	0.09	2	0.08	29	0.50	7	0.23	26	0.47	3	0.30
	31	0.54	5	0.24					25	0.28	6	0.07					18	0.15	1	0.15
	17	0.12	3	0.08					21	0.29	3	0.19								
Mean....	25	0.260	2.1	0.164	27.3	0.380	4.69	0.156	27.4	0.334	4.8	0.144	28.7	0.587	6.5	0.21	22.4	0.222	2.0	0.149

Mr. Houston.

and alkaline content, but also a classification as to its general texture and fertility. With data of this kind as a basis, the prices of the various classes of land can be fixed, and a variable system of payments arranged so that there will be a reasonable possibility of the settler meeting his obligation from the produce of the land.

In the case under discussion, a typical sample of the field notes of the soil investigations is shown in Fig. 8.

A post-hole digger was found to be most satisfactory for obtaining the samples, although several kinds of soil augers were tried. The depth of the hole was usually 6 ft. Samples were taken whenever the soil changed in color or texture. These were packed in 1-qt. glass preserve jars, in order to retain their moisture content, and shipped to the Dominion Chemist for analysis.

The maps were carried in the field in a galvanized-iron case, hinged at the right side and having a beaver-board back. The following abbreviations were used in the soil investigation field notes.

W &.	Very wet.	Water in the hole.
W.	Wet.	Water will drip from sample.
D &.	Very damp.	Water can be squeezed from sample.
D.	Damp.	Slightly soggy.
D—.	Less damp.	Putty-like.
M.	Average soil mixture.	
P.	Dry and powdery.	

AMERICAN SOCIETY OF CIVIL ENGINEERS

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Paper No. 1386

THE YALE BOWL*

By Charles A. Ferry, M. Am. Soc. C. E.

With Discussion by Messrs. Thomas C. Atwood, J. B. French, H. C. Keith, H. F. Dunham, Henry C. Hitt, Alexander S. Lynch, and Charles A. Ferry.

Synopsis.

This paper is descriptive of the new amphitheater constructed for the Athletic Department of Yale University. It is the largest structure of the kind in the world, and provides seating accommodation for nearly 61 000 spectators, a press-stand for about 250 reporters, and a photographers' stand for about 50 operators. It is in the form of a four-center oval, 300 by 500 ft. from face to face of the inner retaining walls, and about 750 by 930 ft. over all, covering an area of about 12½ acres.

The points to be observed in planning the structure were: large seating capacity; ample entrances, in order to avoid congestion at any point; strength and durability, with low cost both for construction and maintenance; and safety, especially for spectators. To meet these conditions, an excavation about 27½ ft. deep was made on the area required for a playing field, and the material thus obtained was used for building a surrounding embankment to a height of about 26 ft. above the general surface of the ground, its inner slope being continuous with that of the excavation.

* Presented at the meeting of November 15th, 1916.

The inner face of the slope, in excavation, was covered with reinforced concrete, built in the form of steps, on which were fastened benches for the spectators. A temporary wood facing, with benches, was built on the inner slope of the embankment, for use until the embankment had become consolidated. The outer slope was seeded with grass. Thirty reinforced concrete tunnels give access, through the embankment, to the spectators' seats; and two tunnels for the accommodation of the players, run directly to the field. The drainage from the playing field and the outer slopes of the embankment is taken by a sewer discharging into a river which runs near the Bowl.

The material was placed in the embankment in 6-in. layers, each thoroughly watered, and then rolled with two rollers, one grooved and the other smooth, one following in the track of the other. Drag-line scrapers, operated from 85-ft. towers running on a four-rail track laid outside the structure, and steam shovels, operating within the Bowl, were used, principally, for building the embankment. The shovels filled bottom-dump wagons which, with the teams of horses, were drawn up the slope by hoisting engines on the top of the embankment. Bottom, side-dump cars, and chutes were used for conveying the concrete from the mixers.

One of the special features of the construction is a cantilever retaining wall, 45½ ft. high, which forms one of the walls of the proposed gate-house.

History.

Football, the most popular of college athletic sports, though one of the oldest, is, under its new rules, one of the most modern of field sports. The first intercollegiate football match in the United States, between Yale and Harvard, was played in 1876, and Mr. Walter Camp, the so-called father of American football, is still one of the most valued advisors of Yale teams.

It was not until 1896, however, that the attendance at Yale games called for special accommodations for spectators at the intercollegiate matches. In that year, the Yale Athletic Association, to provide for any probable future demand for seats, built a stand to accommodate about 18 000 persons. Contrary to expectations, however, this stand soon proved inadequate to meet the demand for seats from those who desired to witness the "big" games, and additions to the stands were

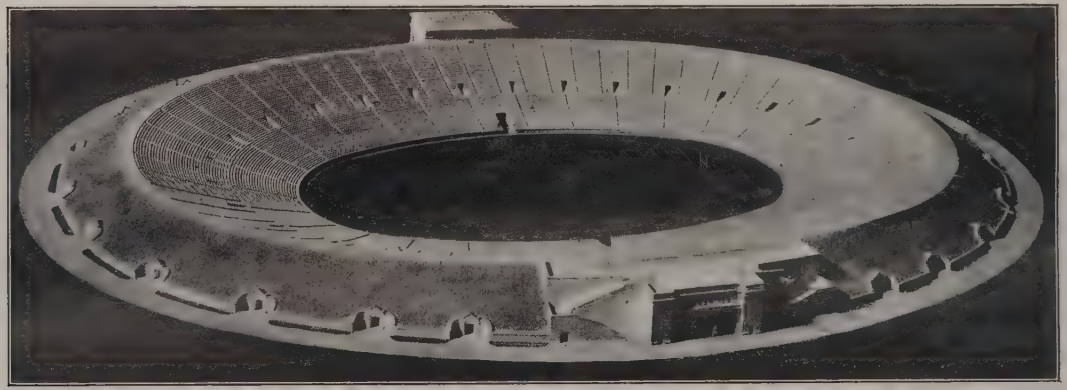
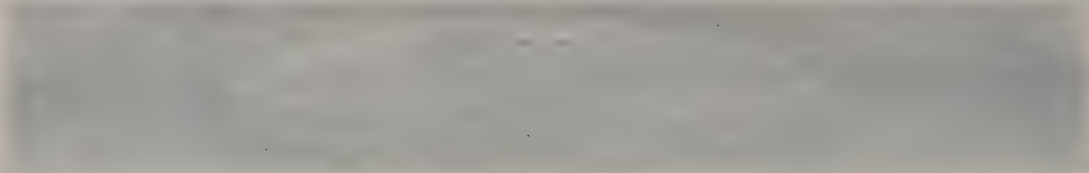


FIG. 1.—MODEL OF THE YALE BOWL.



made from time to time until they finally contained 33 090 sittings and the practical limit of size for a stand in that location was reached. These, so far as the writer knows, were the largest wooden stands ever erected.

In 1904 Harvard erected its steel and reinforced concrete stadium, to accommodate about 23 000 spectators.

As the wooden stands at Yale were not painted, exposure to the weather caused rapid deterioration, and the annual expense for repairs amounted to from $10 000 to $12 000. In addition to this tax, the danger of the destruction of the stands, by fire or otherwise, immediately preceding the date set for a game, thereby necessitating its cancellation, and the thought of the much greater catastrophe of a fire occurring in them during the progress of a game, thereby creating a panic in the great crowd of spectators, caused much anxiety on the part of the management, and, in 1910, resulted in a movement to secure a permanent, more commodious, and fire-proof structure.

The Corporation of the University appointed a committee of seven graduates to consider the whole question of athletic development at the University and the provision of suitable accommodations for spectators at intercollegiate games. This committee recommended an increase in the facilities for out-door athletics, and the erection of a stadium for football; and, on its further recommendation, a committee of twenty-one was appointed to carry the latter into effect.

This committee was organized under the name of "The Yale Committee of Twenty-one, Incorporated", and began to carry its instructions into effect by the purchase of about 87 acres of land near the old athletic field, which, with the latter, gave a total area of about 110 acres available for development, divided into five separate fields by intersecting streets.

Three plans of widely varying type were suggested for the proposed stadium: First, a steel structure composed of separate units mounted on wheels and designed so that these units could be moved in such manner as to provide a stand for either football or baseball as necessity might require; second, a reinforced concrete structure based on the model of the Roman Colosseum, to seat 50 000 persons and designed for football only; and third, the plan suggested by the writer, for a stand constructed of earth faced with concrete, somewhat similar to reservoir construction, from which the idea was derived.

The first plan offered difficulties in the way of providing for suitably rigid foundations, on which the sections of the stand could be moved, without interfering with the playing fields; the second was prohibitive on account of its cost, which was estimated to be about $750 000. The third plan, offering as it did a stable, permanent, and fire-proof structure at a minimum of cost, both for construction and maintenance, was, as subsequently modified, adopted.

The plan as first presented was markedly different in shape from the Bowl as constructed, although the method of construction was the same. As first planned, it provided for football and baseball fields, with a quarter-mile running track, a 220-yd. straight-away track, and spaces for pole vaulting and jumping events, all surrounded by a single stand, an irregular heptagon in shape and of sufficient size to seat 50 000 persons. This stand was estimated to cost about $170 000.

Objection being made to having any part of the baseball field used for football and on account of the possible interference, in practice, by teams in different departments of athletics, the writer was requested to change the plans to provide for football only, and a structure, substantially on the lines of the present Bowl, was designed. This, for a seating capacity of 50 000, was estimated to cost about $235 000. In this plan the outer retaining wall and the tunnels, except the main tunnels to the field, were omitted, the entrances for the spectators being by steps leading over the top of the embankment. With the change in plans to provide for 60 000 spectators, the embankment became so high that tunnels for entrances became desirable, and this necessitated the erection of the outer retaining wall.

During the development of the plans, two questions were prominently raised concerning the probable stability of a concrete structure built on an earth slope varying from 1 on 2½ to 1 on 4: First, whether expansion and contraction, due to temperature changes, and the heaving effects of frost in the underlying sand would not gradually move the concrete down the slope and, eventually, pile it in a disordered heap at the bottom; and second, whether material in the embankment could be compacted so solidly that subsequent settlement would not crack the concrete injuriously.

To answer the first of these questions so far as possible a few simple experiments were made regarding frost action. A sample of the sand was taken from the field within a few hours after a very heavy rain,

and it was found that only about 23% of the volume of the voids was filled with water, the total volume of the voids being about 37% of the volume of the sand when compacted. A bottle, about 10 in. in length and 1 in. in diameter, with a sharply contracted neck, was then filled with sand thoroughly compacted, and with sufficient water added to fill 23% of the voids. The mixture was then frozen, but no indication of any expansion of the mass was noticed. A similar experiment with 50% of the voids filled with water produced a like result. The experiment was then repeated with 75% of the voids filled, and the bottom of the bottle was cracked off.

To ascertain the extent to which water would be retained in the sand by capillary action, the bottle was filled with sand saturated with water. A piece of linen was then tied over the mouth of the bottle which was then reversed and the water allowed to drip. In 2 min. about 16% of the water had run out. The bottle was then set, mouth down, in a bed of sand, the broken off bottom being replaced in order to lessen evaporation from that end. After 24 hours 67% of the water had run out, and 80% at the end of 8 days. It was concluded, therefore, that, as the slope was to be entirely covered with concrete, thereby excluding all water except such as could pass through the expansion joints, the danger of any disintegrating effect resulting from frost in the underlying sand might be neglected.

As the steps were to be built in the form of horizontal arches, it was thought that these would take care of any possible movement due to temperature changes.

To provide for a possible settlement of the embankment, it was suggested that the steps be built in the form of reinforced concrete slabs, somewhat after the plan in the Harvard Stadium, supported on reinforced concrete girders resting on concrete piers built into the earth, the girders being raised a few inches above the face of the slope. Then, in case of any settlement, the girders could be shimmed up on the piers, and the structure restored to its original form. The estimated cost of this plan, however, proved to be so much in excess of that for concreting directly on the earth that it was abandoned, and it was decided to concrete first only the portion of the slope in excavation, and to provide the face of the embankment, temporarily, with a wooden superstructure, until such time as the embankment had ceased to settle.

Description.

The Bowl, as constructed, has a playing field, oval in plan, 300 by 500 ft., varying from a true ellipse by about 6 ft. at the point of greatest divergence, the radius of the sides being 420 ft. and that of the ends 120 ft.

The old wooden stands had been built with straight sides and ends, the corners being filled in with sections at 45° to these lines. The Har-

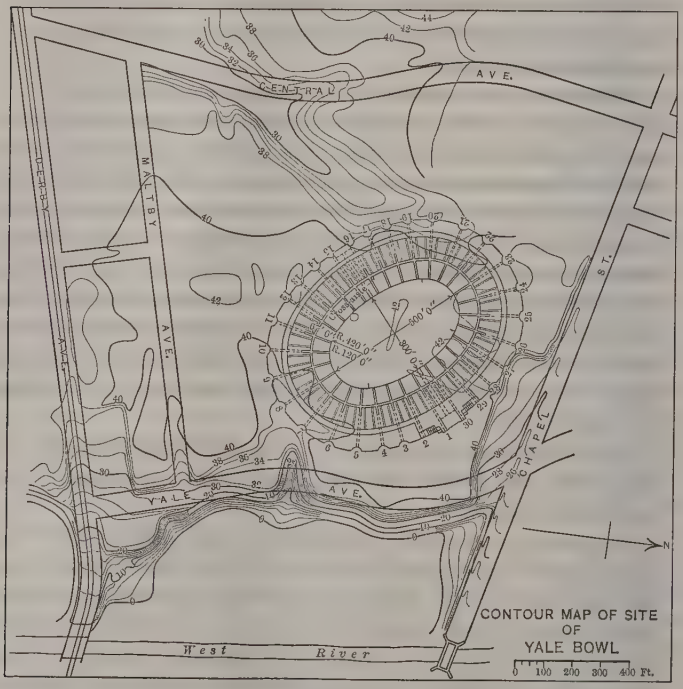


Fig. 3.

vard and Princeton Stadia have straight, parallel sides, joined by a curved section at one end, the other being left open. In such stands a person occupying a seat on a side near one end cannot get a good view of plays on the same side of the field near the opposite end. The oval form was adopted for the Bowl in order that all the spectators might have an unobstructed view of the plays in whatever part of the field

they might occur. This form also enables each spectator to obtain a view of the entire audience, a very thrilling spectacle when all the seats are filled and every one is roused to a high pitch of excitement by some brilliant play.

The space between the outer lines of the gridiron and the surrounding wall is of sufficient width to permit of a running track, 20 ft. wide and of a length only 200 ft. short of ¼ mile. As there was not room for a 220-yd. straight-away track—a feature which is considered necessary for an athletic field—without tunneling under the seating section, the track feature has been abandoned for the Bowl. The playing field is laid out so that the 5-yd. lines point toward the sun at 3 P. M. in the middle of November, the time when the principal match games are played, thus making the lighting of the two ends of the field the same.

The earth was scooped out over the area to be occupied by the playing field and piled up in an embankment surrounding it, the inner slope of the latter being continuous with that of the inner face of the excavation. The depth of the excavation was about 27½ ft. below the general surface of the ground, and the top of the embankment about 26 ft. above it, making the total elevation from the surface of the field to the top of the fill 53½ ft.

The inner face of the slope was graded to a smooth surface, with vertical as well as horizontal curves, to serve as a foundation for the sixty concrete steps. The tread of the steps is uniformly 30 in., with the exception of the step at the inner portal of the tunnels, which is 52 in. wide and is used as a cross-aisle. The bottom step is 4 ft. above the playing field, and the height of the steps increases uniformly by 0.006 ft. from the bottom riser, which is 8 in. high, to the top one, which is 12⅛ in. high.

The dishing shape, formed by this combination of vertical and horizontal curves, suggested the name "Bowl" which has been given to the structure, and provides advantages to the spectators for viewing proceedings on all parts of the field not possessed by any other large stand. The vertical curve gives a person occupying the top seat the same facility for seeing over the heads of the persons occupying the seats in front of him as that possessed by the spectators in the lower rows, the clearance being practically the same in all the rows of seats from a point about 200 ft. from the inner retaining wall.

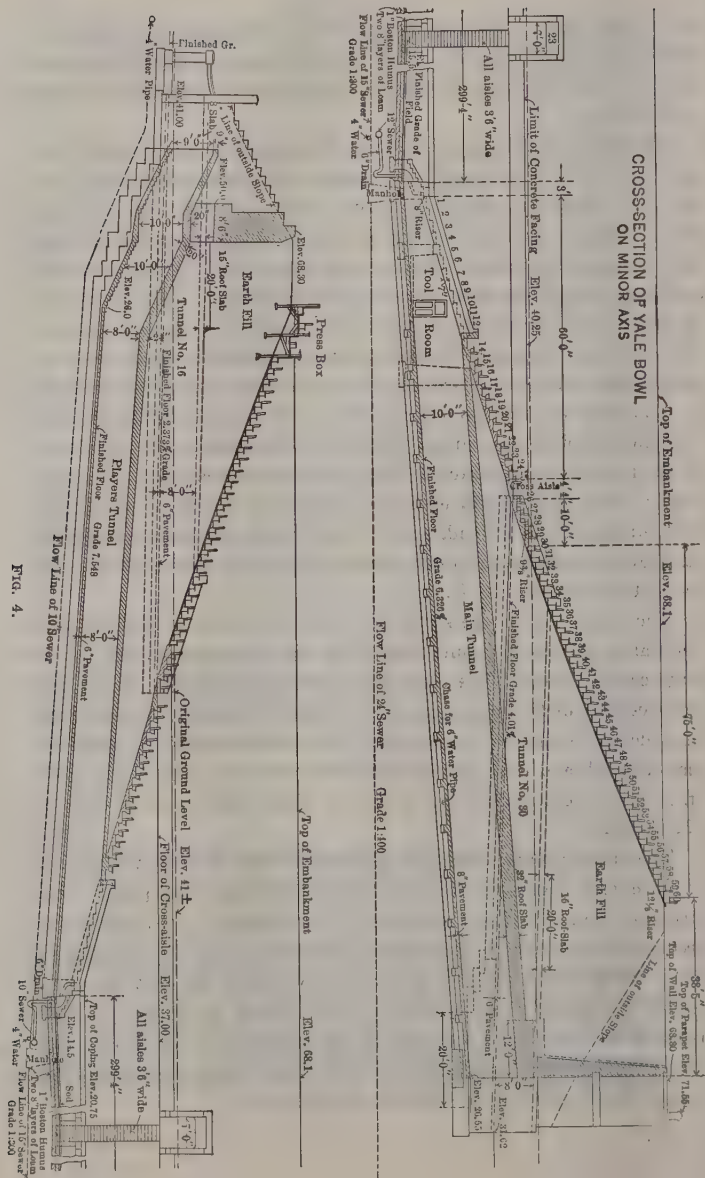

CROSS-SECTION OF YALE BOWL
ON MINOR AXIS
Top of Embankment
Elev. 68.1
Earth Fill
Tool Room
Main Tunnel
Tunnel No. 30
Flow Line of 24" Sewer
Grade 1:400
Limit of Concrete Facing
Elev. 40.25
All aisles 3'6" wide
Finished Grade of Field
Finished Floor
Grade 6.520%
Chase for 6" Water Pipe
Finished Floor Grade 4.01%
Top of Parapet Elev. 71.55
Top of Wall Elev. 68.30
Line of outside Slope
Finished Gr.
Water Pipe
Elev. 41.00
Elev. 68.30
Press Box
Earth Fill
Tunnel No. 16
Finished Floor 2.373% Grade
Players Tunnel
Finished Floor
Grade 7.548
Flow Line of 10" Sewer
Original Ground Level Elev. 41±
Floor of Cross-aisle Elev. 37.00
All aisles 3'6" wide
Top of Coping Elev. 20.75
Elev. 14.5
Flow Line of 10" Sewer
Grade 1:500

FIG. 4.

The embankment is 15 ft. wide on the top in order to allow room for a promenade, 12½ ft. wide, back of the top row of seats, and then slopes down to a retaining wall, the top of which is 9 ft. above the surface of the ground outside the Bowl. This slope is 1 on 2, except opposite the portals, where it had to be steeper on account of the recessing of the portals into the embankment.

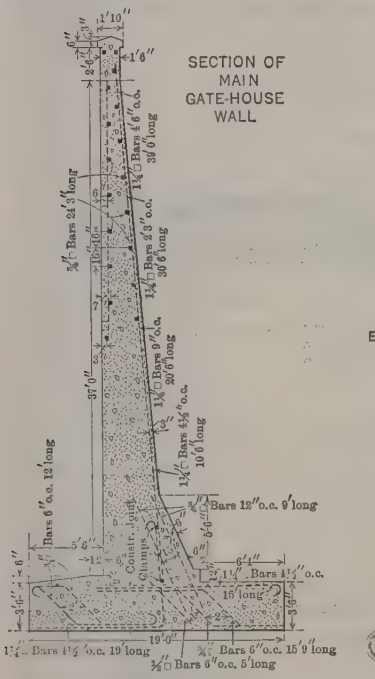


FIG. 5.

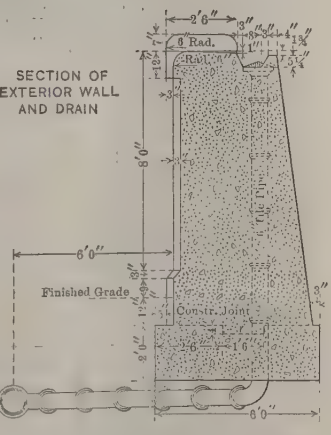


FIG. 6.

RETAINING WALLS.

The playing field is surrounded by a retaining wall, 4 ft. high, its top being level with the tread of the bottom step of the seating section. This wall is surmounted by a parapet which, with a 12-in. coping, is 27 in. high. The outer retaining wall is finished with a coping 12 in. thick, the front, top corner of which is rounded to a curve with a 6-in. radius, in order to render the wall more difficult to scale, in case any one de-

sired to take a short cut, financially, into the Bowl to witness a game. The pilasters project 3 in. from the face of the wall. They are carried up 6 in. above its top, and are finished with a cap 30 in. square over the front portion of the wall. For convenience in construction, both retaining walls were made as chords instead of arcs or circles, the angles being at the pilasters, which also serve as expansion joints. The outer portals are recessed 9 ft. into the embankment, and are joined to the main wall by curved wing-walls.

UNDER SIDE OF ROOF SLAB TO TOP OF FILL.	ROOF SLAB AT CENTER *D*	□TW. ROOF RODS *A1 B1 B2*	SIDE WALLS	VERT. RODS *A2*
16′ to 34′	15″	⅞″	15″	⅝″
6″ " 16′		⅞″	15″	⅞″
Less than 6′	8″	½″	15″	½″

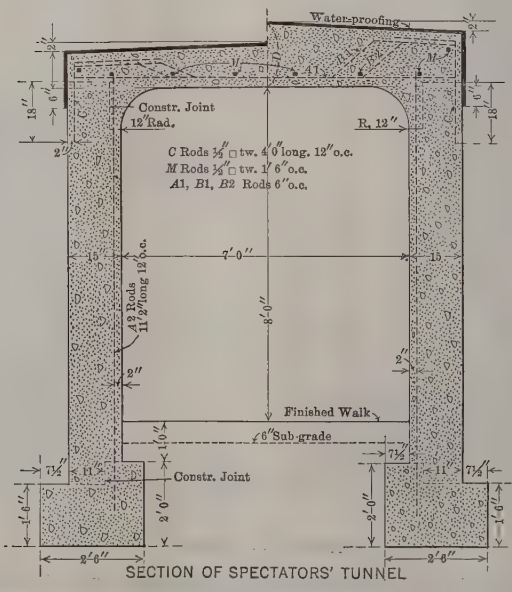


FIG. 7.

TUNNELS.

Thirty tunnels through the embankment are provided as entrances to the amphitheater for spectators. These are all 7 ft. wide and 8 ft. high. They start from the level of the ground outside the Bowl and

terminate at the twenty-fifth row of seats from the bottom, where they are spaced uniformly along the cross-aisle. As the level of the ground opposite the main gate-house is 11 ft. lower than that on the opposite side, these tunnels are all on a slope, those on the westerly side being down toward the field on a maximum grade of 2.37%, and those on the easterly side up on a maximum grade of 4.18 per cent. The end tunnels have grades of 0.24 and 0.26%, respectively, just sufficient to afford good drainage.

UNDER SIDE OF ROOF SLAB TO TOP OF FILL	ROOF SLAB AT CENTER	TW. ROOF RODS A3 B3 B4	SIDE WALLS	VERT. RODS A4
	ft			
23′ to 32′	32″	1½″	15″	⅞″
14′ to 23′	to	1″	15″	¾″
7′ to 14′		⅞″	15″	⅝″
3′ to 7′		¾″	15″	½″
Less than 3′	12″	⅝″	15″	½″

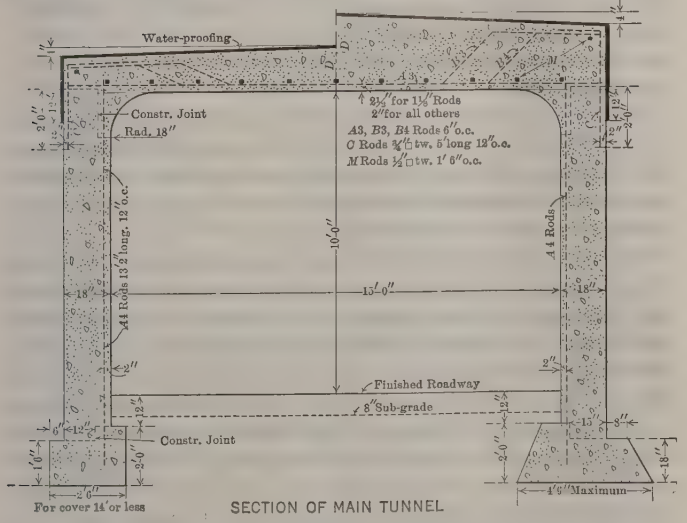


FIG. 8.

In addition to the thirty spectators' tunnels, there are two for the players, which can also be used as exits for the spectators. These run from the level of the ground outside the Bowl directly to the playing field. The one on the easterly side, 15 ft. wide and 10 ft. high, affords facilities for carting in loads of hay or straw, when necessary to protect the field from frost. It has a grade of 6.33 per cent. The other tunnel is 10 ft. wide and 8 ft. high. The difference in elevation between

the portals of this tunnel being too great to admit of a straight ramp for its entire length, a double flight of steps, with an intermediate platform 6 ft. wide, was built at the outer end and then a ramp, on a grade of 7.55%, for the remainder of the distance to the field.

The footings of the tunnels were all made monolithic, but the remaining portion was divided into lengths, not exceeding 30 ft., by expansion joints. Each joint was made by inserting in the side-wall forms steel plates bent to form a tongue and groove with flaring sides. On one side of these plates was placed a strip of tarred felt and, as the forms were filled, the plates were drawn up out of them, leaving the felt embedded in the concrete, thereby preventing the bonding of the adjoining sections. This device enabled the contractor to complete a tunnel at one pouring, instead of being obliged to wait two or three days for the concrete in one set of sections to set before pouring the intermediate ones.

The sides and roof of each tunnel are reinforced with steel and, down to a line just below the underside of the roof, are water-proofed with three layers of roofing felt and one layer of Tartex, all cemented together and to the concrete with roofing pitch. On top of this a 1-in. protective coating of cement mortar was laid.

The sides of the tunnels, below the water-proofing, and the backs of the retaining walls, were mopped with hot roofing pitch. Before the pitch was applied, all rough places on the surface were smoothed over with mortar, and the whole surface was then washed over with a thin grout, to insure the closing of all "pin holes". That this treatment was efficacious in producing a water-tight wall was accidentally demonstrated. Between the end-walls of the large gate-house and two of the adjacent tunnels are spaces about $2\frac{3}{4}$ ft. wide. The sand in these pockets became covered with concrete and grout, spilled from the chutes during the process of building the walls, which effectually sealed the interstices between the particles of sand, and rendered the pockets practically water-tight. Water from the roof of the tunnels ran into these pockets and, at one time, stood about 6 ft. deep without showing any signs of leakage or even sweating inside the tunnels.

The floor of each tunnel has a granolithic pavement. In the largest tunnel this is 8 in. thick; in all the others it is 6 in.

All the tunnels are well lighted by electricity, there being five 100-watt lamps in each of the spectators' tunnels and six in each of the

others. Hemispherical depressions were made in the ceiling, from which lamps project down into Holophane globes suspended from the under side of the roof.

Aisles.

An aisle, 52 in. wide, extends around the entire amphitheater at the inner portals of the tunnels. From this other aisles run down, opposite each portal, to give access to the lower twenty-five rows of seats, and a flight of steps leads from the lower end of each of these aisles to the playing field. Other aisles, one on each side of each spectators' tunnel, afford access to the upper thirty-five rows of seats and to the promenade on the top of the embankment.

Grading.

The surface of the playing field is graded to the form, approximately, of a segment of a football, the ground at the center being 1 ft. higher than that at the foot of the steps. On the narrow strip, 5 ft. wide, next to the retaining wall, the grade is broken so as to drain to the basins under the steps. The field was first graded to a sub-grade 18 in. below the established grade. Black loam, 16 in. deep, was then spread over the field in two layers, each layer being rolled with a 5-ton roller. Boston humus, 1 in. deep, was then spread over the loam and harrowed into it to a depth of about 3 in., and the entire area was then turfed. A thin layer—about $\frac{1}{8}$ in.—of sifted black loam was then spread over the turf, and grass seed was sowed and raked in. Although the turfing was done during the hot summer, and the foundation of the field was clean sand and gravel, judicious watering caused the grass to begin to grow and, in a few weeks, a firm field with a strong turf was obtained.

No sub-soil drains were laid under the field, as the material is very porous, and the level of the ground-water, as indicated by a well in the vicinity, is about 12 ft. below the surface of the field, thereby insuring good sub-soil drainage.

The outer slope of the embankment was brought to a sub-grade 10 in. below the established grade, and then filled with black loam. Strips of turf, 1 ft. wide, were then laid, 8 ft. apart, parallel to the edge of the promenade. The remainder of the slope was then seeded to grass. The strips of turf were intended to prevent the slope from gullying in

case of heavy rains before the grass was sufficiently grown to afford the necessary protection.

Besides the building of the embankment, there was a large amount of grading around the Bowl to make the grades of the surrounding grounds conform to the neighboring streets. In addition, Yale Avenue, a street about 1 500 ft. long and 80 ft. wide, was graded, the maximum cut being about 13 ft. The material from this grading, which was not required in the embankment, was used for filling about 11 acres of marsh, to a depth of about 3 ft., rendering it available as playing fields.

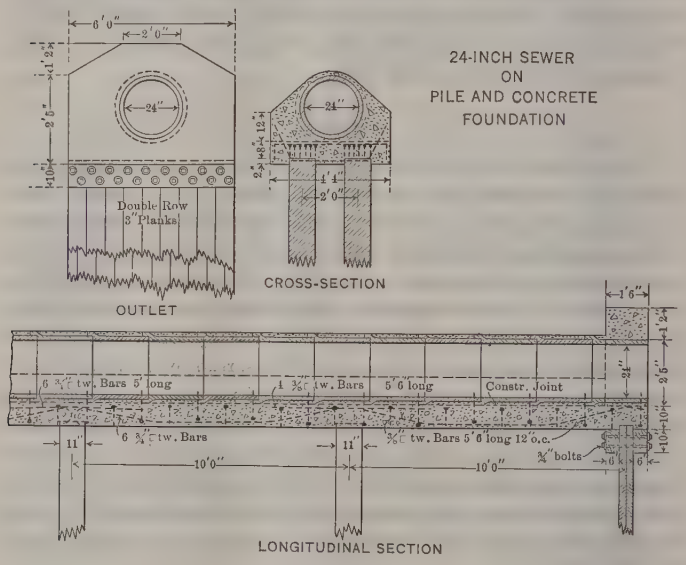


Fig. 9.

Drainage.

The water falling on the inner slope of the amphitheater is caught in a gutter formed in the top of the retaining wall just in front of the first riser. This gutter is 6 in. wide, and varies in depth from 6 in. at the center of the panels to 9 in. at the aisles. It is covered with a galvanized cast-iron grating. Drain pipes, 6 in. in diameter, run from this gutter, through the wall, and discharge on slightly dishing slabs of concrete built under each of the flights of steps which lead from the aisles to the playing field. In the center of each of these slabs is a

small receiving basin, covered with a grating, which connects, by a 6-in. drain, with the drain which encircles the field. These basins also receive the drainage from the field.

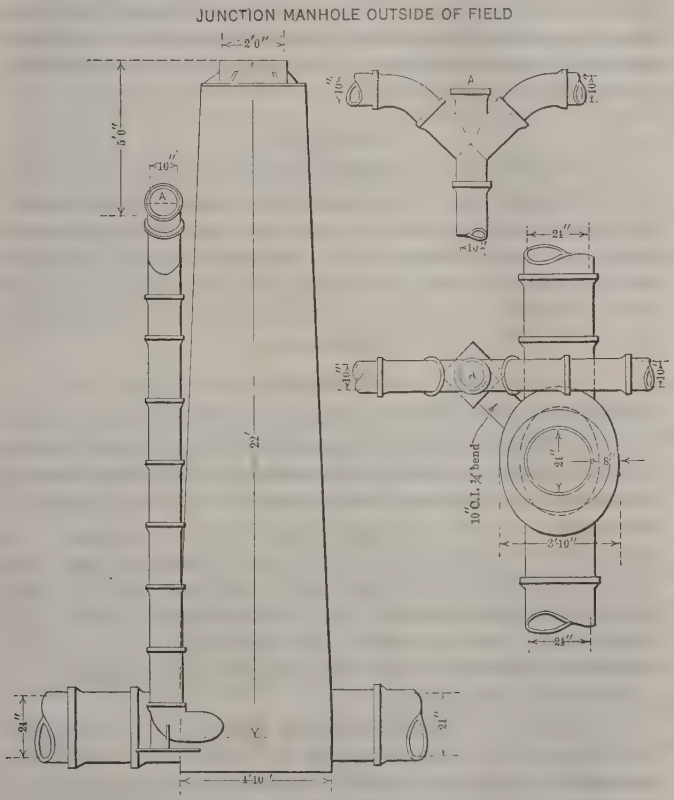


Fig. 10.

The water which falls on the outer slope of the embankment is collected in a gutter formed in the top of the retaining wall near the inner edge. This gutter is 9 in. wide and 6 in. deep, with flaring sides, to lessen the risk of the wall being cracked by ice, and is connected, at the lower end of each section, by a 6-in. drain with the drain which runs around the outside of the Bowl.

The field drain connects with a 24-in. pipe which runs under the main tunnel, and the drain which encircles the structure connects with the same conduit, by a drop-manhole, just outside the front wall of the proposed main gate-house. The main drain discharges into West River, that portion which crosses the marsh being laid on a reinforced concrete slab supported by piles. The capacity of the drains is based on a rainfall of 2 in. per hour.

Water Supply.

To provide facilities for watering the field, the outer slope of the embankment, and the surrounding grounds, there are flush hydrants at frequent intervals around the playing field just inside the inner retaining wall, and outside the Bowl in the portal recesses. These are connected with 4-in. mains laid parallel with the retaining walls both inside and outside the structure, and are supplied with water from the mains of the New Haven Water Company.

Gate-Houses.

Gate-houses, which will contain showers, dressing-rooms, etc., for the teams, will be built at the entrances to each of the large tunnels. Owing to shortage of funds, it was decided to postpone the erection of these buildings until after the completion of the seating portion of the structure. In order to complete the embankment, however, it was necessary to construct the back walls of each of these buildings. The wall of the smaller house is 38 ft. high in its highest part. By using the end and proposed partition walls of the building for buttresses, and by the use of a small quantity of steel reinforcement, a wall was designed which was much lighter than would have been necessary if a gravity section had been built.

The height of the highest portion of the wall for the main gate-house is (including a 3-ft. parapet) 45½ ft. For this wall there was no opportunity to obtain any lateral support from other walls, and a cantilever wall was constructed, this type being adopted in preference to a counterfort wall on account of convenience in placing and consolidating the back-filling, although, probably, the latter would have been somewhat cheaper. This is, so far as known to the writer, the highest wall of the cantilever type ever constructed.

The main reinforcing rods in this wall are 1¼ in. square. For the lower quarter of the height the rods are 3 in. apart on centers; for the

second quarter, 6 in.; for the third quarter, 12 in.; and for the top quarter, 24 in. As the wall was so high that it was impracticable to make these rods in single lengths and extend them down into the footing, they were made in two pieces with an allowance of fifty diameters for a splice just above the top of the footing. The rods were so large and the spacing so close at the bottom that it was deemed advisable to place them in the splice, side by side, on a line at right angles to the direction of the wall. To avoid the danger of the wall being split by the eccentric strains which might be developed by this arrangement, and to assist the concrete in transferring the strain from one set of rods to the other, three clamps were used on each splice, one each at top, bottom, and middle. Tests at the Mason Laboratory of Yale University showed that, with two clamps, a pull of 10 000 lb. was required before the rods would slip, thereby relieving the concrete of so much of its work.

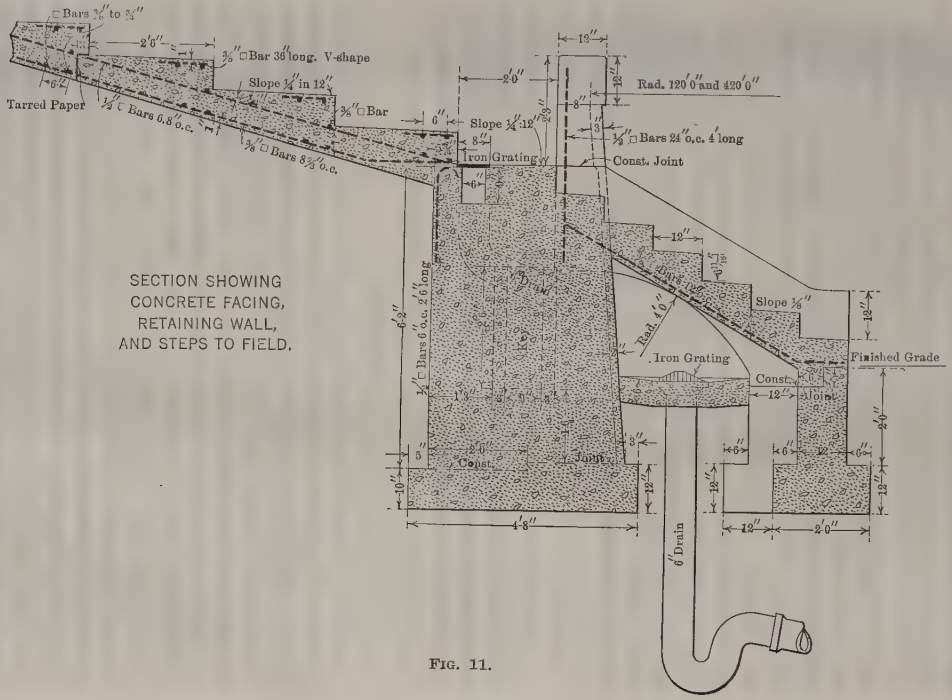
Tool-Rooms.

Rooms for the storage of hose, tools, etc., required in the care of the field and grounds, were made in the embankment, one on each side of the approach to the main tunnel. These rooms are 24½ by 12½ ft., and of varying height, the concrete seating steps forming the ceiling. The entrance doors to the rooms open on the approach to the tunnel.

Concrete Facing.

The concrete facing on the portion of the amphitheater which is in cut was built in sections three steps wide and of lengths varying from 3½ to 17½ ft., depending on the location. Each block is reinforced with steel in both directions and at top and bottom, sufficient reinforcement being used to permit the blocks to sustain a live load of 50 lb. per sq. ft., whether they are supported at the sides, ends, or middle. The blocks, at the circumferential joints, overlap 3 in. and are separated by a layer of tarred felt. All radial joints are butt-jointed, and were given a coat of roofing pitch, to prevent the bonding of the blocks, but in the middle joint in each panel there was inserted a layer of Elastite, ⅜ in. thick, to provide for expansion.

The facing was built in alternate blocks, laid checker-board fashion, 3 days being allowed for the setting of the concrete in one set of blocks before the intermediate ones were laid. The blocks forming the aisles were laid last, as they were the shortest. The slabs were all 7 in. thick



SECTION SHOWING CONCRETE FACING, RETAINING WALL, AND STEPS TO FIELD.

FIG. 11.

in the thinnest part, under the junction of the tread with the riser. In addition to the regular reinforcement, a strip of wire lath, 6 in. wide, was built into each of the aisle steps, ½ in. below the surface, in order to prevent the spalling of the corners of the steps.

To prevent the glare from such a large surface of concrete, if left in its natural color, and to afford variety of coloring, the facing was made a dark slate by the addition of 1 lb. of lampblack to each barrel of cement used in the granolithic surfacing. The inner retaining wall was left in the natural color of the concrete.

Wood Facing.

The wood facing, on the slope of the embankment, consists of 2 by 6-in. sills, laid radially not more than 4 ft. apart. To these were spiked triangular-shaped blocks, 3 in. thick and 18 in. long on the upper edge, for supporting the tread of the steps, which are 18 in. wide, the remainder of the 30-in. space allowed for each row of seats being left uncovered. Although this left the earth slope exposed to the elements, the material in the embankment is so porous and absorbs water so readily that little trouble has been experienced from washing of the embankment.

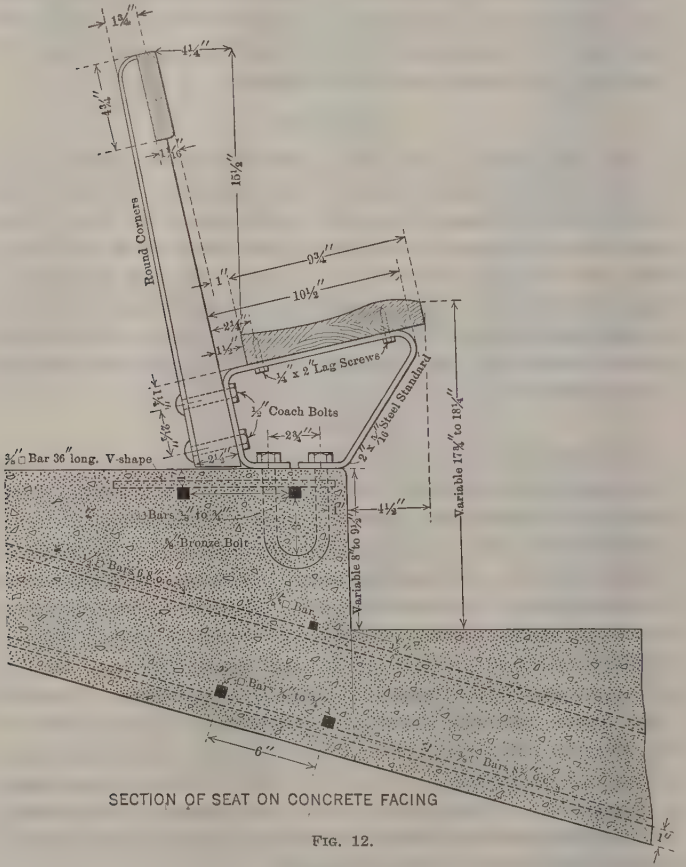
Seats.

The permanent seats built on the concrete facing consist of 2 by 10-in. planks, with the upper side planed to a curved surface, supported on galvanized-steel standards anchored to the concrete. These standards were made in four different sizes, in order to fit the varying height of the risers and make the height of the seat within the limits of 17¾ and 18¼ in. The back rest is a 1¼ by 5-in. board, with the front face curved and the corners rounded, supported by 2½ by 2½-in. wooden standards bolted to the backs of the steel ones. The wood for the permanent benches is rift-sawed Douglas fir from Oregon. U-shaped Tobin bronze bolts were specified for anchoring the steel standards to the concrete, but a lower price being offered by the contractor for Sherardized-steel anchor-bolts, the first benches erected were anchored with them, but, as these proved to be not well adapted for the purpose, their use was abandoned, and about two-thirds of the seats are fastened with the bronze bolts as originally planned.

The temporary benches on the wood facing consist of 1¼ by 10-in. spruce boards supported by 2 by 12-in. wood standards spiked to the

sills. These benches are provided with back rests of the same kind as the permanent seats.

A minimum spacing of 17¾ in., with an average of a little more than 18 in. was allowed for each sitting. With this spacing there are 60 617 seats for spectators.



SECTION OF SEAT ON CONCRETE FACING

Fig. 12.

Press-Stand.

A stand for the representatives of the Press was erected on the westerly side of the Bowl, just below the top of the embankment.

This is provided with two rows of seats, and shelves for convenience in writing or telegraphing. With a spacing of 21 in., this stand will accommodate 249 persons, sitting. In addition to this, there is pro-

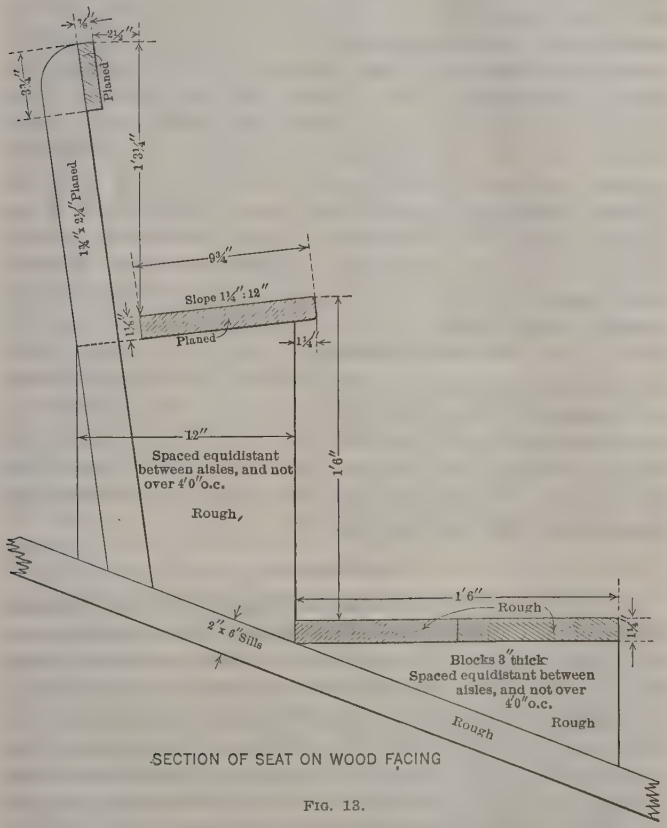


FIG. 13.

vision for three rows of persons, standing, giving accommodations for about 400 additional, or a total of about 650 in the press-stand. Back of the press-stand, and raised a little above it, a stand for photographers will accommodate about 50 operators.

Peculiarity of Seating Arrangements.

The seating arrangements in the Bowl are noticeably different from those in other large stands, such as the Harvard, Princeton, and Syracuse Stadia. The plan of having a low riser for the bottom step and then increasing uniformly to a high one at the top is peculiar to this structure, and has the advantage that, while it gives the spectators occupying the upper rows of seats the same facility for seeing over the heads of those sitting in front of them which the occupants of the lower rows enjoy, it permits of a lower and, consequently, a much cheaper and more stable structure. For example, in the Harvard Stadium the 31 rows of seats occupy a height of about 41 ft.; in the Syracuse Stadium the 18 rows of seats have a height of 27 ft.; in the Princeton Stadium the 48 rows have a height of about 62 ft.; but, in the Bowl, the 60 rows of seats have a height of only 49½ ft. In these other stands, the height of the riser is the height of the seat, the spectator sitting directly on the concrete step or on a board laid directly on it. Seats thus constructed are not very inviting on a cold day, particularly during a rain storm, when the water is dammed back on the seats by the spectators sitting on them. In the Bowl, the benches are raised a sufficient height above the concrete to permit the water to run freely over the steps without coming into contact with the spectators. The Bowl seats, also, are provided with back rests, but those of these other structures have none.

Construction.

Ground for the Bowl was broken on June 23d, 1913, by President Hadley, of the University, turning the first sod. The real construction was begun during the following month by first removing the loam from the site and from the adjoining area which required grading. The black and yellow loam was stored in separate piles and saved for resurfacing the grounds after the completion of the structure. The average depth of this preparatory stripping was about 22 in. For removing the loam, on short hauls up to about 100 ft., drag-scrapers were used; on hauls from about 100 to 250 ft., wheel scrapers; and for long hauls, bottom-dump wagons.

Several plans for placing the material in the embankment were considered by the contractor, but the one finally adopted for handling the greater part of it was the use of two drag-line scrapers running on

adjustable cables and operated from two towers, 85 ft. high, by two 12 by 16-in. double-drum engines, supplied with steam from two 125-h.p. locomotive boilers. The towers ran on a four-rail elliptical track laid around the outside of the Bowl. One end of the carrying cable was fastened to a "deadman" on the opposite side of the field from the tower; the other end ran over a pulley at the top of the tower and was fastened to one of the drums of the engine, by which means it could be regulated for filling or emptying the bucket, the cable being slackened for filling the scraper and then tightened for dumping it. The second drum was used for hauling the scraper up the slope; it ran back to the filling point by gravity.

As the two scrapers, working night and day, did not remove the material rapidly enough to indicate the probable completion of the contract within the time specified, two $\frac{3}{8}$-yd. rotary steam shovels were started, in order to expedite the work, and these filled bottom-dump wagons. The teams with their loads were then drawn to the top of the embankment by a portable hoisting engine working there, and the material was distributed as desired.

As it was impracticable to roll the embankment below the level of the tops of the tunnels, dependence for packing the material in this portion of the work was placed on such consolidation as could be obtained by the liberal use of water, and such tamping as was done by the teams being driven over it. Streams of water were always kept playing on the embankment where material was being deposited, and the portions next to the tunnels and the retaining walls were generally sluiced into place from piles deposited by the teams or scrapers.

Above the tops of the tunnels, the material deposited by the scrapers was left in ridges extending across the embankment; that from the wagons was left in scattered piles; in each case sufficient material was placed to make layers 6 in. deep when leveled down. The leveling was done with horse-drawn scrapers. The material was thoroughly watered and then rolled, first with a grooved and then a smooth roller, one following in the track of the other. The rollers weighed 800 lb. per ft. of length, and, by using them alternately, the effective weight was practically doubled. Each roller was drawn by four horses, and each passed four times over the entire area being rolled. The embankment was kept about 1 ft. wider than the theoretical lines in order to insure the compacting of the material out to the edges of the finished fill.

To ascertain the settlement in the embankment, after its completion thirty-one benches were established around the top. Each bench consisted of a mass of concrete in which was inserted a ½-in. square steel rod, placed about 4 ft. below the surface, the end of the rod terminating a few inches below the surface. Levels taken on these benches have shown that, after a lapse of 2 years, the average settlement has been only about ¼ in., and the maximum settlement at any bench about ½ in. About one-half of this settlement occurred within the first month after the establishment of the benches.

A comparison of the volume of the embankment with that of the excavation shows that the material in the former is nearly 7% more dense than that of the natural bank. It is probable, therefore, that the slight settlement which occurred was caused by the compression of the underlying material, due to the added weight of the fill, rather than by any subsequent consolidation of the embankment itself.

The favorable results obtained in securing a solid embankment were probably due to the character of the material of which it is composed and the method of placing it. The material was sand and gravel, almost entirely free from clay, loam, or other substance which would retain water. By the alternate use of the two kinds of roller, not only was a downward pressure exerted, but, at the same time, a lateral motion was given to the particles of sand, thereby tending to bring them into close contact with one another.

Water for sprinkling the embankment and mixing concrete was obtained from a group of driven wells at the foot of the bluff, at the edge of the marsh, and was distributed by a 4-in. pipe, laid around the outside of the Bowl, with hose connections at frequent intervals. The water from the wells was pumped by a single-acting, triplex pump, operated by an electric motor. The contractor was required to furnish at least 150 gal. of water per min., and this quantity was delivered continuously, day and night, including Sundays and holidays, while the embankment was under construction. A connection was also made with the mains of the New Haven Water Company, in order to insure a constant supply in case of a failure of the wells or a break down of the machinery.

As the drag-line scrapers left the inner face of the slope in a very irregular condition—a series of deep gullies and sharp ridges—the contractor devised a leveling scraper for smoothing up the work. This

consisted of a frame, about 6 ft. square, of 12 by 12-in. timbers, the front faces of which were shod with steel plates. This frame was attached to the drag-line by a bridle, and was drawn up the slope in the same manner as the excavating scrapers. The final trimming of the slope was done by hand shoveling.

Two mixers were used for supplying concrete for the tunnels. For the footings, there was a portable, ½-yd., gasoline mixer at the inner portal of each tunnel. This delivered its material into cars running on a track laid between the footings, from which the concrete could be dumped into the forms on either side as desired. An electrically operated ¾-yd. mixer near the center of the field was used for mixing the concrete for the portion of the tunnels above the footings. The concrete was conveyed on cars running on an industrial railway track laid on the ground from this mixer to a point near the inner portal of the tunnels, then up an inclined trestle to their top and then on timbers laid on the side forms. From the mixer to the foot of the incline the cars were hauled by a horse, and for the remainder of the distance by a hoisting engine outside the Bowl.

For building the outside retaining wall and the smaller gate-house walls, there were two ½-yd. mixers on an elevated trestle built on the site of the gate-house. These delivered the concrete into bottom, side-dump cars running on an industrial railway laid on top of the wall forms. For the large gate-house walls, a mixing plant was established near the wall, and the concrete was delivered into a 120-ft. elevator, from which it was distributed by chutes. The same plant also made the concrete for the inner retaining wall and the facing steps. For this work the concrete was delivered by the chutes into the bottom, side-dump cars running on a portable track laid just above the inner portals of the tunnels; from them it was distributed, by portable chutes laid on the ground, directly into the forms, the slope of the bank being steep enough to cause the concrete to flow in the chutes. The greatest distance through which concrete was carried by the chutes, in a single run, was about 250 ft.

For handling sand and stone at the two main mixing plants, elevated bins were constructed so that cars or trucks could be run on top of them and deliver material without shoveling. Chutes at the bottom of the bins delivered material into cars which were divided by partitions to hold the required quantity of each ingredient for one

batch. From the cars the materials were dumped directly into the mixer.

All the sand for the work was obtained at the site. Though an abundance of gravel, of a size suitable for the concrete, was found in the excavation, only a small quantity of it was used in some of the footings, as it contained a considerable proportion of sandstone pebbles which it was feared would absorb moisture and disintegrate under the action of frost if exposed to the elements. Most of the stone used was crushed trap rock. This was brought from a quarry about 2½ miles distant by two motor trucks: a 5-ton truck which not only carried its own load but also hauled 3 tons in a trailer; and a 3-ton truck. The price paid for hauling the stone was 27 cents per ton.

The cement used in the work was furnished by the contractor, but was paid for as a separate item, thereby enabling the engineer to change the proportions of the ingredients in the concrete, as seemed desirable, without causing any dispute as to the quantity used. The cement was all tested at the mill by agents of testing laboratories.

Comparison with Other Stadia.

The Bowl is believed to have the largest seating capacity of any structure of the kind in existence, and to have been exceeded by only one of ancient times—the Circus Maximus of Rome, which is stated to have seated 380 000 persons. Statements regarding the capacity of the Roman Colosseum, the most widely known of the ancient amphitheaters, vary widely from 45 000 to 100 000. It is stated that in the ancient amphitheaters only the classes were provided with seats, the masses being obliged to stand.

With the seats removed, the Bowl could accommodate comfortably a standing audience of about 125 000. The Athens Stadium seated 50 000; the stadium of the College of the City of New York seats about 10 000; the Syracuse and Tacoma Stadia each accommodate about 20 000; the Boston baseball stands, 24 000; the double-deck stands at the New York Polo Grounds, about 28 000; the Harvard Stadium, as first constructed, seated 23 000, but, with additions and with temporary seats at the open end, it can accommodate about 45 000; the new Princeton Stadium seats about 41 500; and the Bowl about 61 000.

Though the Bowl is the largest of the modern stands, it is also, probably, the cheapest per sitting. Though the actual cost of other

stands is not accurately known, the contract price, as reported in engineering and other journals, gives the average cost per sitting as ranging from about $36.00 for the stadium of the College of the City of New York down to about $8.50 for the Princeton Stadium. The cost per sitting for the Bowl, as constructed, including the grading of the grounds around it, was about $7.35. The cheapness of the Bowl was partly due to the exceptionally favorable conditions at the site for building this kind of a structure. This was a level plain elevated about 40 ft. above a marsh from which it was separated by a steep bluff, thus affording perfect drainage at a minimum of expense. The material was clean sand and gravel extending to an unknown depth, easy to handle, and ideal for making a solid embankment.

Previous to the building of the Bowl, the materials used in the construction of grand-stands were wood, steel, reinforced concrete, or a combination of the two latter, and, in the case of ancient structures, masonry. Though wood is a suitable material for stands of small size, it is entirely unsuited for large ones, owing to rapid deterioration when exposed to the weather and liability to destruction by fire. Though monumental structures of masonry, like the Roman Colosseum, were possible with the slave labor available in the first centuries of the Christian era, it would be impracticable to finance such an undertaking at the present time. Steel and reinforced concrete are, from an engineering standpoint, more practical building materials for structures of this kind.

Maintenance charges are an important factor to be considered in planning for a steel stand of large size. Whether temperature changes and frost action will injuriously affect skeleton structures of the size of the large modern stadia, when built of reinforced concrete, either alone or in combination with external steel, is an interesting question yet to be answered. Apparently, the only portions of the Bowl which will require material expense for maintenance are the benches, as the remainder of the structure is practically a portion of the earth's crust—a low hill surrounding a small valley.

One advantage of this form of construction, besides its cheapness, is the facility with which the plastic materials, earth and concrete, may be adapted to the formation of curved shapes, thus, without any material expense, permitting of curved instead of straight sides, which were such an objectionable feature of the old wooden stands at Yale.

Another advantage is that a stand can be increased in size without materially affecting any part of the work already done. By building a balcony over the promenade and the portion of the seats above the tunnel portals, the seating capacity of the Bowl can be increased to about 100 000 without affecting any portion of the structure other than by the removal and replacing of a few blocks of concrete to permit the building of proper footings for columns, the embankment being sufficiently stable for the support of the additional structure.

With nearly 61 000 seats for spectators provided, it was thought the stand would be large enough to meet any possible demand for accommodations from those interested in football, but when the time limit expired for the receipt of applications for tickets for the first game in the Bowl—Yale *vs.* Harvard in 1914—it was found that more than 80 000 had been received. To meet this demand, so far as possible, extra seats were erected over the promenade, so that the number of people who witnessed that game was probably a little more than 70 000. In spite of this vast number, in 12 min. from the close of the game, the Bowl was practically empty of people, and this time might have been shortened by about 5 min. had not the spectators tarried to witness the celebration of the victory, on the gridiron, by the partisans of the winning team. Although the Bowl was emptied in such a short time, there was no congestion at any point except in the aisles, as the capacity of the tunnels to handle the crowd was greater than that of the aisles, and the thirty-two tunnels distributed the crowd over such a large area, the inner circumference of which was a little more than ½ mile, the distance around the outside of the Bowl, that there was plenty of room for the free movement of the people.

Adaptation for Other Purposes.

Although the Bowl was built for football alone, its remarkable acoustic properties render it admirably fitted for the production of dramatic events on a large scale, and two such have already been held there with pronounced success: the Greek play, "Iphigenia in Taurus", given by Mr. Granville Barker under the auspices of the Yale Dramatic Association, and the opera "Die Walküre", by the Metropolitan Opera Company, under the auspices of the Yale School of Music. For these performances, the stage was erected near the center of the field, and a

section of about 25 000 seats was set apart for the spectators at one end of the amphitheater.

To celebrate the two-hundredth anniversary of the removal of Yale College to New Haven, a pageant is now being prepared in which it is expected that about 8 000 performers will participate.

Under favorable atmospheric conditions, when there are only a few people in the Bowl, a whisper spoken at the center of the field can be distinctly heard at the tunnel portals; and a conversation, in an ordinary tone of voice, can be carried on between persons stationed on opposite sides of the amphitheater. At the performances mentioned, even the softest passages could be plainly heard in the most remote seats reserved for spectators. With the seats removed, a person with a good voice and accustomed to public speaking could, undoubtedly, address an audience of 125 000 and be heard distinctly by every one.

The following are some of the principal items of construction:

Earthwork	331 000 cu. yd.
Mass concrete	16 000 " "
Concrete facing	111 000 sq. ft.
Wood facing	145 000 " "
Granolithic pavement	35 000 " "
Cement	26 000 bbl.
Steel for reinforcement	482 tons.
Wood benches	18 miles.
Turf laid	161 000 sq. ft.
Sewers	6 400 lin. ft.
Water mains	4 700 " "
Area covered by structure, about	12½ acres.

The Bowl was built by the Yale Committee of Twenty-one, Incorporated, Mr. Thomas DeWitt Cuyler, Chairman, with Mr. David Daggett, Chairman of the Structures Committee. Edward G. Williams, M. Am. Soc. C. E., was Advisory Engineer, and Mr. Donn Barber, Consulting Architect. The writer was the Designer and Engineer in charge of construction. Mr. Joseph H. Mulvey was Chief Inspector. James B. French, M. Am. Soc. C. E., served as Consulting Engineer for a few months at the beginning of the work, and Thomas C. Atwood, M. Am. Soc. C. E., as Construction Manager for a few months at the end. The Sperry Engineering Company, of New Haven, Conn., was the contractor for the whole work.

DISCUSSION

Mr. Atwood.

THOMAS C. ATWOOD,* M. AM. SOC. C. E.—As has been stated in the paper, the Yale Bowl is the largest structure of its kind which has ever been built. When compared with the designs for other stadia proposed for this place, its simplicity and effectiveness at once command attention. The character of the structure, essentially of earthwork, ensures its permanence; and its safety from fire and accident is beyond question.

The original sketches called for a structure, the estimated cost of which was somewhat less than $400 000, but the several engineers and architects engaged on the work have advised certain changes, affecting the appearance and permanence of the structure, which have nearly doubled this cost.

The paper is hardly fair to other stadia in stating the cost of the Bowl as $7.35 per seat, this being the cost in its present incomplete condition. When finally completed the cost will be about $750 000, or approximately $12.30 per seat, or considerably more than the cost of the Princeton Stadium. This includes the cost of completing the concrete lining of the Bowl and placing the permanent seats, the gate-houses, permanent toilet facilities, and permanent fence. The Princeton Stadium is not fully completed, the wooden seats being lacking, but the final cost will probably not be more than $10 per seat. Of course, the difference in size must be taken into account, and a larger structure of the Princeton type would undoubtedly cost more per seat. All things considered, it is probable that the Bowl is the more economical structure for the location, which is ideal for its type, and for the size demanded.

There is no doubt that the shape of the Bowl, with its curved sides, is far better than the regular stadium U-shape with straight sides. It is this horizontal curve of the sides, rather than the vertical curve produced by making the height of each riser uniformly greater than that of the one next below it, which enables the rise per seat to be small, the straight-sided structures requiring more rise in order that spectators may see all parts of the field. The importance of this is seen when it is realized that in the Bowl only 20 000 of the seats are enclosed between the goal lines extended, and that more than 40 000 are at the ends, back of the goal lines. At Princeton about 16 000 seats lie between the goal lines, and 25 000 are back of them, but only one end is enclosed, the other being open.

Of the problems in connection with the Bowl, few were of less interest than the eleventh-hour proposition to put in a track and provide a 220-yd. straight-away course by tunneling at one end. Careful

* New Haven, Conn.

estimates showed that this would involve such heavy construction that the cost would be about $50 000, and would detract considerably from the appearance of the Bowl. Experience with such tunnels, as at Syracuse, N. Y., has shown that the running conditions are not satisfactory; and, as the Committee had plenty of land available, and as an ideal track on open ground, together with a permanent stand to seat 10 000 people—as many as are likely to attend the intercollegiate track events—would cost little if any more than the tunnel, it was decided to complete the Bowl as planned and build a track and stand later. Mr. Atwood.

Referring to the general construction: when the speaker was placed in charge, on January 1st, 1914, approximately one-third of the work had been completed, and the embankment had reached the top of most of the tunnels. As noted in the paper, the rolling could not be carried on between the tunnels, and it seemed improbable that the bank had been thoroughly consolidated at all points, so the top of the bank was leveled up, divided into sections, and flooded, holes being drilled down through it at the same time by iron bars or water jets about 8 ft. apart, on centers, both ways. At some points no settlement could be observed, but at others there was considerable. On this uniformly compacted base the remainder of the bank was built, as described in the paper, and with gratifying results.

The whole settlement of the completed bank is not shown by the bench-marks noted in the paper, as these were not set until the bank had been completed for nearly 2 months, but there is no doubt that the settlement was very small compared with that found in other carefully built banks, such as those under aqueducts.

The water pipe entering the Bowl was carried through the main tunnel in a small trench in the concrete floor, so that a break could not affect the foundations of this tunnel and the embankment.

The turf strips, noted as being placed in the outside loam slope, were very successful in preventing deep wash, as was shown by a torrential rain which occurred before the seeded area had started to grow, the water in no case getting under the sod, and no gullies more than 3 in. deep being formed.

The high reinforced concrete retaining wall mentioned has performed its work well, a movement outward at the top of little more than $\frac{1}{2}$ in. being the greatest noted, or corresponding to a settlement at the toe of the wall of only about $\frac{1}{8}$ in. Relative to this matter, it may be mentioned that claim for royalty on account of the Bone patent has been made within the past year, and the speaker has done considerable work in investigating this, with the aid of searches made in the Library of the Society, and has some references to walls, built before the date of the patent, which are not included in the summary of this subject contained in *Engineering News*.

Mr. Atwood.

In the concrete facing inside the Bowl, the horizontal arching action was not made use of, dependence being placed on the interior retaining wall to care for any possible tendency of the blocks to slide down hill.

The lampblack placed in the mortar for the top of the facing blocks aided materially in giving them uniformity of color, as the sand used contained uncertain quantities of iron which caused some of the concrete to have a pinkish hue.

The aisles were given a top dressing of Trus-Con floor hardener, and no wear is yet perceptible; but it is difficult to say whether or not this is due to the hardener. This hardener should be applied with care by skilled men, as a few blocks, which apparently got an overdose, have turned rusty.

The inner retaining wall and the tunnel portals inside the Bowl were originally planned to have pipe-rail fences, but this was changed to heavy concrete parapet walls, at a considerable saving in cost and with a decided improvement in the appearance of the Bowl, the general idea of massive simplicity being carried out more fully.

The design of the permanent seats was another interesting problem. As finally worked out by Mr. Everard Thompson, Assistant Secretary of the Committee, and the speaker, the seat was placed on the edge of the step and supported on a steel bracket or standard which allows full play for wind and water, so that the structure can be cleaned with comparative ease.

The lumber for these seats was selected with great care. It was desirable to use edge grain lumber, and investigation showed that this could be obtained in only three woods, Douglas fir, redwood, and Western hemlock and Noble fir classed together. As redwood is likely to stain clothing when wet, and Western hemlock could not be obtained in sufficient quantities, Douglas fir was chosen.

Time was of great importance, and some doubts were expressed as to the possibility of getting the lumber in the 3 months available. The order was given by the speaker on August 18th, and the first carload arrived on September 29th, just 6 weeks later. When the order was given, the lumber was in the log. It was sawed, kiln-dried, planed, and shipped within 2 weeks, and was 4 weeks on the road. The kiln-drying, in this case, was preferred to open-air drying, as it boiled out the pitch pockets in the fir. All the lumber is vertical grain, clear and free from knots or other defects, except an occasional pitch pocket.

Another interesting matter was the design of the playing field. The underlying sand was so porous that a stream from a 2-in. hose would all disappear within 2 or 3 ft. of the end of the hose, so the problem was to retain moisture enough to prevent the grass from burning out, and yet have a field that would dry quickly after a rain. This was accomplished by using 18 in. of black loam and by crowning the field 1 ft., the cross-section consisting of two very flat curves, convex upward,

joined by a sharper curve at the center of the field. The scheme has worked perfectly. Mr. Atwood.

The summary of items of construction given is for the Bowl in its present state, and, of course, many of these will be changed when the work is completed.

In order to assist any one desirous of obtaining further information, the following bibliography of articles on the Bowl, all of which are illustrated, is given:

Engineering Record, Unsigned articles in March and November, 1914.

Engineering News, Article by the speaker, November 12th, 1914.

Yale Alumni Weekly, Article by the speaker, November 27th, 1914.

Proceedings, Connecticut Society of Civil Engineers for 1915. Paper by the author.

Journal, Boston Society of Civil Engineers, June, 1916. Paper by the speaker.

Much credit is due to the Committee of Consulting Engineers which assisted in getting the work properly started. This Committee consisted of Professor John C. Tracy, of Sheffield Scientific School, and H. C. Keith and James B. French, Members, Am. Soc. C. E. Later, Mr. French was appointed Consulting Engineer in charge of the design and construction of the work, and the first contract was prepared under his direction.

The credit for carrying on the remainder of the work is largely due to Edward G. Williams, M. Am. Soc. C. E., who served his Alma Mater without pay in the capacity of Advisory Engineer, and by his wise advice and careful supervision brought the work to a successful conclusion. The architectural features of the Bowl were designed by the Consulting Architect, Mr. Donn Barber, of New York City, and are exceptionally effective.

The three contracts covering the work described were finally concluded early in 1915. The speaker, however, was retained by the Committee to carry on the engineering work, and contracts were prepared covering the completion of the seating structure, permanent toilet buildings, and permanent fences, in expectation of proceeding soon with this construction. This expectation was abandoned in the spring of 1916, and the only work now being done is the construction of some 17 000 temporary seats for the Yale-Harvard game, on November 25th, 1916. This will make the total seating capacity of the Bowl on that date 77 189.

J. B. French,* M. Am. Soc. C. E.—The speaker can most properly speak for those who advised against building the concrete facing directly on the embankment. Professor John C. Tracy, of Mr. French.

* New York City.

Mr. French. the Yale Faculty, H. C. Keith, M. Am. Soc. C. E., and the speaker were asked to report on this question in November, 1912. Plans submitted to this Committee at that time were materially different from those followed in the construction. As described in the papers then submitted, the surface of the inner slope was to be cut into steps and faced with concrete 4½ in. in thickness, laid directly on the sand, in the same manner as concrete curbs and sidewalks are built. No steel reinforcement was proposed, and all drainage was to be carried from the top of the embankment down over all the steps to the level of the playing field. At this time, also, it was hoped that the Bowl could be completed in the fall of 1913. In other words, it was proposed to build a continuous concrete lining, nowhere more than 4½ in. thick, to cover an area measured in acres, on a new and hurriedly built embankment 27½ ft. high. It was the opinion of the Committee that, in a structure as important and conspicuous as this was bound to be, the procedure proposed would invite undesirable defects, and that, though it might be possible to compact the embankment so that unsightly cracks might not occur, it was preferable to reduce the risk to a minimum. The Committee, therefore, though approving the general plan, reported adversely to the proposed construction of the concrete lining, and recommended a lining, lifted a few inches above the ground, made up of inverted channel-shaped slabs, supported by radial girders resting on small pedestals built in the embankment. It was proposed to make the slabs uniformly 30 in. wide, of the same cross-section, and to lay out the work so that both slabs and girders should have spans of about 20 ft., and, to the greatest extent possible, be duplicates of each other. The variation in the rise of the steps or benches was to be taken care of by the form of the tops of the radial girders. It was planned to have the very large number of slabs required cast in advance in a concrete casting yard, where the concrete work could proceed independently during the progress of the earthwork, and to erect the pre-cast units very quickly with a small traveling derrick as fast as the embankment was completed.

To facilitate the construction, and at the same time take care of the rainfall on the inner lining, the slabs were not to fit each other closely, but were to be separated by slits at least ½ in. in width and were to be tilted to shed water. Rainfall was to be prevented from accumulating, and was to be allowed to escape directly through the sand and gravel, as it had always done before in the plateau in which the structure was to be built. The ability of this material to absorb rainfall had been proved to the satisfaction of the Committee by sinking a shaft at the site from the surface of the ground to below the depth of the playing field, and this shaft had cleared itself of water immediately after every heavy rainfall. To protect the newly made inner

face of the earth structure from being carried down by the drainage, it was proposed to cover it with a layer of cinders, about 6 in. thick, a method found particularly effective in the construction of railroad embankments. Mr. French.

The Committee obtained estimates on the construction described from contractors most experienced in such work, and was convinced that the cost of the completed structure would not be increased materially by the proposed changes from the original plan.

The recommendations of the Committee were formally adopted; detailed plans and specifications were prepared and accepted; and a contract was executed for the earthwork and concrete tunnels, leaving the slab work for the interior lining to be included in a later contract. At this stage, however, other counsel prevailed, and the slab and pedestal plan was abandoned. From the description given in the paper, it appears, however, that when the time arrived for actually building the concrete lining on the embankment, notwithstanding the extremely careful and expensive methods used to compact the fill, it was not deemed wise to disregard the advice originally given, and the construction of the permanent lining was postponed until the danger of settlement should be known to have passed. It is also to be noted that the temporary construction finally placed on the embankment allows the rainfall to pass directly into the underlying sand and gravel without accumulation, exactly as was proposed in the slab and pedestal plan.

In view of the final decision, not to build directly on the embankment, it seems unfortunate that so much expense was incurred in compacting the material, as this will now be done by several years of exposure to the elements.

It is the opinion of the speaker, and he knows it to be the opinion of others who have had experience in building railroad embankments, that the rolling of clean sand and gravel, such as was available for this work, is of doubtful advantage, and, on the other hand, that the liberal use of water while the material is being deposited is highly effective. From the levels reported by the author, it seems to have been demonstrated that the fill was well compacted, but it does not seem at all clear that this result was due, to any great extent, to rolling. As the paper states, the very considerable part of the embankment between the tunnels was not rolled, and as, in addition to the water applied while the material was being deposited, the bulk of the fill had been exposed to a winter of snow and rain, the actual advantage of the rolling does not seem to have been very conclusively demonstrated.

H. C. Keith,* M. Am. Soc. C. E.—What the speaker is inclined to say is not so much in discussion of the paper as an expression of Mr. Keith.

* New York City.

Mr. Keith. appreciation of the great idea which Mr. Ferry presented to the world in building the stadium in this form, and in this way, at New Haven. There are very few places where such a stadium could be built satisfactorily: the nature of the soil and the location with respect to the possibilities of drainage were great factors in making this a possibility.

Although it was necessary for the speaker and his associates on the Board of Consulting Engineers, at one time, to differ from Mr. Ferry in his recommendations in some details, yet they always felt that he was deserving of great credit for the plan presented. The method of construction has also been an important factor in making the design a success.

It is disappointing that the paper gives no information as to the cost of the stadium as it is finished to-day; it is hoped that before it is printed in its final form, such figures will be included. It is well that the *Transactions* of the Society should make ample record of this construction.

Mr. Dunham. H. F. Dunham,* M. Am. Soc. C. E. (by letter).—The double rolling, with frequent disturbance of the material, was helpful in some degree in making a firm embankment. Experience has shown the advantage of vibration in solidifying sand and gravel when under pressure. Those who have had occasion to direct the tamping of loose sand and gravel for railway water tanks and building foundations will be apt to agree in this. The vibration in all such cases, even when slight, is more effective than the pressure. This was well illustrated by experiments made a few years ago in New York City by Mr. Alfred O. Crozier in compacting sand for concrete products. The almost instant reduction in volume and hardening of the mass that occurred under slight vibration in two directions and without any pressure except the weight of the material itself was a matter of surprise and interest to all observers. In the writer's opinion, any easily operated tamping machine, even if heavy blows are not delivered, would be more effective than the rollers. This view is not to be considered as a criticism of this excellent paper, which brings distinction to its author.

Mr. Hitt. Henry C. Hitt,† Assoc. M. Am. Soc. C. E. (by letter).—The writer has been very much interested in this particularly well written description of an unusual piece of construction. The author has given a very complete and satisfactory description, full of the data one wants, of the details of the design and of the construction methods.

The design certainly appears to be very successful. The "bowl" shape is an outdoor adaptation of old principles of auditorium design, and undoubtedly provides the most ideal seating scheme, as well as being architecturally beautiful. It is strange that it has not been

* New York City.

† Seattle, Wash.

Fig. 14.—The Tacoma Stadium.

used more frequently with other stadia. The slate-colored concrete, with a trim of the inner retaining wall and the tunnel portals in natural color, the comfortable seats, the beautiful elliptical shape, and the careful orientation as to the afternoon sun, must all go to make the structure a credit to every one responsible. Mr. Hitt.

The author, in comparing the Bowl with other modern stadia, states that it is probably the cheapest per sitting unit. In this, however, he is in error, as the Tacoma Stadium, which the Bowl resembles in some ways, has cost, completely equipped, only about 85% of what the Bowl, in its unfinished condition, with a portion of the seats temporary, and the dressing rooms not built, is reported by the author to have cost, per unit of seating capacity.

Thus far, the total cost, of every kind, of the Tacoma structure, built in 1910, has been $147 000, and an estimate of the seating capacity, based on the same assumptions as used by the author, is 23 784, giving a unit cost of $6.20. The architects claim a seating capacity of 32 000, and more than 35 000 people have actually occupied the stadium at one time, with none of them on the field, but part of them standing.

The site has also a reserve capacity of about 15 000, as the sodded terraces rise for 43 ft. above the top of the seats around more than half the perimeter, at a proper slope for future seating as the needs of the rapidly growing city demand it. The addition of these seats would materially cut down the unit cost of the whole.

The two structures were designed to fill widely different conditions, but are similar in the result. The fields are of practically the same area, but where the Yale Bowl was designed strictly for football and seating capacity, with few or no limiting conditions, the site of the Tacoma Stadium was limited, and the field had to be adapted for the general use of the people and the adjoining High School, for track work, football, baseball, pageants, concerts, or political meetings. This made a complicated problem, which was most happily solved.

The seating scheme is similar to one end only of the Yale structure, with every occupant able to see the whole field, and most, if not all, the audience. At Tacoma, however, the designers had a natural location, of which they took the fullest advantage. The site is on a promontory in the very heart of the city, with the playing field 140 ft. above the harbor. From the Stadium seats one gets a view probably not equalled by that from any similar structure. Spread before the spectator is a magnificent sweep of busy Tacoma Harbor, green hills, and snow-clad mountains. The Stadium is flanked on each side by beautiful buildings rising above the seats, on one side the Stadium High School and on the other the State Historical Museum.

Although the site was naturally somewhat adapted to the purpose, there was a very large quantity of excavation, more per unit of seating

Mr. Hitt. capacity than with the Yale Bowl, and about two-thirds of it proved to be very hard cemented gravel. All the seats are carried on columns and radial girders about 20 ft. apart, the risers of the seats, 3½ in. thick, reinforced with a structural frame carrying the seat load between the girders. Part of the columns, at the outer end, are carried on piling, and part of the field at this end is a fill of 147 ft., but there has been no trouble from settlement.

Twenty-eight arc-light projectors are placed around the field above the seats, and twelve more are suspended by cables, high above the field, affording ample light for evening performances. The acoustics are very fine, and concerts have been entirely successful. The stadium is fully equipped with dressing rooms, and comfort stations for the spectators.

The principal quantities were, approximately:

Earthwork	185 000 cu. yd.
Reinforced concrete	10 000 cu. yd.
Reinforcing steel	160 tons, plus
Clinton fabric	130 000 sq. ft.
Area of field	3.6 acres

The Tacoma Stadium was built by the Tacoma School Board, Mr. Frederick Heath having been the architect, and Mr. L. A. Nicholson the engineer.

Mr. Lynch. Alexander S. Lynch,* Assoc. M. Am. Soc. C. E. (by letter).—Mr. Ferry states that Sherardized bolts were first used to fasten the seats to the concrete and then abandoned for the bronze bolts, as originally planned. He does not state why this change was made, other than that the Sherardized bolts were unsatisfactory. The writer would like to know why they were unsatisfactory.

Mr. Atwood states that the greatest movement in the high walls was ½ in. A recent examination of the walls shows that the movement in one was less than ¼ in., and in the other it was ¾ in., making a noticeable and very unsightly crack. The writer would like to ask if the soil under both walls was the same, or if there was a difference in the design of the walls.

The final cost of the bowl as given by Mr. Atwood is $750 000. Does this include the cost of the 23 000 seats erected for the Harvard games of 1914 and 1916, and subsequently removed?

There seems to have been no apparent reason for applying Trus-Con floor hardener to the aisles. Mr. Atwood states that no wear is perceptible. Neither is any wear perceptible in the floors of the tunnels, although the traffic there is much greater. There was no more reason for applying Trus-Con hardener to the aisles of the Bowl than for apply-

* West Haven, Conn.

ing it to the concrete sidewalk. The only effect now noticeable is that the hardener was improperly applied, leaving many of the blocks a dirty, rusty brown, unpleasant to look at, and spoiling the general soft gray tone produced by the use of lampblack. Mr. Lynch.

After reading Mr. Atwood's discussion, one is led to wonder why Mr. Ferry should have submitted a paper on the Bowl. Mr. Atwood gives "much credit" to the Committee of Consulting Engineers, which assisted in getting the work "properly started", and to others, but nowhere does he mention Mr. Ferry, except to refer to a paper read by him before the Connecticut Society of Civil Engineers. The writer, like hundreds of other engineers throughout the country, was under the impression that Mr. Ferry had not only designed the Bowl, but that he had in truth created "something new under the sun." The writer still holds that belief.

CHARLES A. FERRY,* M. AM. SOC. C. E. (by letter).—In reference to the cost of the Bowl, Mr. Atwood states: "when finally completed the cost will be about $750 000, or approximately $12.30 per seat." He does not give the details of this estimate, further than to state the different pieces of work required to complete the structure. Of these, the plans for the "permanent toilets" and the "permanent fence" have not been finally settled, so far as the writer knows; consequently, the cost might be any sum that Mr. Atwood chose to assume. It is questionable whether the cost of a fence enclosing the Bowl should be charged to that structure any more than that of the land on which it is built. Mr. Ferry.

The sum paid to the contractor was a little less than $448 000. The plans for the Bowl were radically changed after the work was well advanced, as mentioned by Mr. French, thereby necessitating much extra work, as well as extra prices for considerable work already under contract. This also included a large sum for grading, which, although desirable in the development of the plans for the athletic field, was not necessary in the construction of the Bowl itself. It also included the cost of the temporary dressing rooms and the erection and removal of the temporary toilets and extra temporary seats for the Yale-Harvard game of 1914. If the extra temporary seats built for the Yale-Harvard games are to be included in the cost of the Bowl, as apparently they have been in Mr. Atwood's estimate, then, in all fairness, their number should be included in computing the cost per seat. On this basis, that cost would be only about $9.70 per seat, even at the extravagant estimate of $750 000.

The writer is confident that if the Bowl had been built as originally planned by him, the cost of the completed structure, including that of permanent toilets, which was not included in the original esti-

* New Haven, Conn.

Mr. Ferry.

mate, would not have exceeded $450 000, or about $7.40 per seat, as the contractor's bid, based on the engineer's list of quantities, was lower than the writer's estimate made on the same basis.

This price compares favorably with that quoted by Mr. Hitt for the Tacoma Stadium, namely, $6.20, when the relative sizes of the two structures are taken into account, as the price per seat increases with the increase in size of the structure, regardless of the materials used.

Judging from Fig. 14, illustrating Mr. Hitt's admirable discussion, benches are not provided in the Tacoma Stadium, the spectators sitting directly on the concrete, as in the Harvard and Princeton Stadia. The permanent benches for the Bowl cost about $0.93 per seat, including painting.

The cost of the Tacoma Stadium would seem to confirm the writer's opinion that large stands constructed in the manner used by him in building the Bowl are more economical than those built by any other method hitherto used.

The history of this structure well illustrates the truth of the old adage that "too many cooks spoil the broth." Only, in this case, the "broth" (Bowl) was not spoiled, but the "high cost of living" (building) was very largely increased.

The writer is exceedingly skeptical regarding the utility of Mr. Atwood's method of consolidating the embankment, as described by him. No levels were taken on the embankment at the time, and, as there were no near-by permanent objects with which to compare the height of the embankment, the measurement of the settlement was dependent on the imagination of the observer. To believe that a given quantity of water, discharged at the bottom of small holes, 2 or 3 ft. deep and about 8 ft. apart (assuming that such holes existed in the saturated sand after the pipes or bars were withdrawn), would have any more effect in consolidating material several feet below the bottom of the holes, than an equal quantity percolating uniformly through the full depth of the sand, requires more credulity than is possessed by the writer.

As for the effect on the upper portion of the embankment by jetting water down into it, the writer is positive that the result was just the opposite of that desired. The water injected into the embankment washed out the finer particles of sand and deposited them in flat, volcano-like cones around the pipe, and the coarser particles were left in as loose a condition as it was possible to make them by the boiling action of the water escaping upward around the pipe. Only about one-half the circumference of the Bowl was treated by this novel method of "consolidating" material; and the opinion that this work was not only useless, but worse, would appear to be substantiated by

the fact that the benches which showed the greatest subsequent settlement were in the sections thus treated, and the points of least settlement were in those portions which did not receive the "water cure", although the latter were on the west side of the Bowl where the depth of fill, between tunnels, was greatest, the tunnels on the east side being largely in cut. Mr. Ferry.

Mr. Atwood states that:

"In the concrete facing inside the Bowl, the horizontal arching action was not made use of, dependence being placed on the interior retaining wall to care for any possible tendency of the blocks to slide down hill."

The writer has never had any fears that the concrete blocks would slide down hill, particularly those near the foot of the incline, where the slope is only about 1 on 4, unless pushed down by those near the top, where it is about 1 on 2½. If, however, there ever should be any such tendency to slide, he has much more faith in the resistance due to the arch action than in that of the inner retaining wall, for the concrete steps abut against the thin lip of the gutter formed in the top of this wall, and, although the lip is reinforced, the pressure required to shear it off is, probably, only a small fraction of that which would be required to crush or buckle the concrete facing under arch action. At the present time, 2½ years after the laying of the concrete on the lower part of the slope, there are no indications that there has been any sliding of the blocks down hill.

The experience of the past 2 years indicates the wisdom of the Committee in abandoning the plan for an open concrete facing raised above the embankment, as described by Mr. French, and in re-adopting the plan for a tight facing laid directly on the earth, as recommended by the Advisory Engineer, Mr. Williams.

Although the material of which the embankment is composed is very porous and readily absorbs the water from an ordinary rainstorm, it does not take care of that from heavy downpours, such as are likely to occur several times a year. Thus far, there has not been much trouble from the gullying of the inner face of the embankment, as the runs between the seats are in contact with the ground, thus forming water-breaks every 30 in. Nevertheless, after a very heavy rainstorm, the care-takers are obliged to shovel sand from the runs and fill up the gullies which have formed between them. If gullies, 3 or 4 in. in depth, will form in one storm when there are water-breaks every 30 in., it is reasonable to suppose that serious ones, such as might undermine the concrete footings, would form in a period of a few years when the water had an unobstructed run of 150 ft., even though the slope was covered with a layer of cinders.

Mr. Ferry. The principal point in controversy between the writer and the engineers consulted by the Committee previous to the adoption of the plans, was whether the material in the embankment could be consolidated so that there would be no settlement which would be likely to cause injurious cracking of the concrete facing. The writer's 20 years' experience in back-filling sewer trenches and, particularly, in the construction of an embankment on which to build a slow sand filter, led him to believe that it was possible, by the use of water, supplemented by thorough rolling, to construct a stable one for the Bowl. For the filter foundation, the maximum depth of the fill was about 16 ft. The material composing the fill was a mixture of clayey hardpan and a very fine sand saturated with water—practically quicksand. These materials were mixed roughly by working both borrow-pits at the same time, but there was no special effort to secure a thorough mix by harrowing. No water was used, except such as was contained in the quicksand and from occasional rains. The material was leveled off in layers about 8 in. in thickness, and was thoroughly rolled with a grooved roller; a smooth roller was not used. The fill was completed late in the fall. Benches were established at various points, after which an embankment, about 18 in. high, was built around the edges of the fill, and the area was flooded with water, the pond being maintained throughout the winter. In the spring, the water was drawn off, the benches were tested, and it was found that the maximum settlement was about $\frac{1}{2}$ in., a little better result than was obtained at the Bowl, although, at the latter, the depth of fill was nearly twice as great as at the filter foundation.

The argument advanced by Mr. French, that because "the bulk of the fill had been exposed to a winter of snow and rain, the actual advantage of the rolling does not seem to have been conclusively demonstrated," does not apply to the section of the embankment, about 300 ft. in length, in the vicinity of the proposed large gate-house.

A portion of this fill, that immediately back of the retaining wall, is more than 40 ft. in depth (the deepest fill on the work) and all of it, from foundation to promenade, was deposited in the 9 weeks between April 4th and June 6th. The material was consolidated only by the use of water and rolling, as specified for the remainder of the work, the assistance rendered by Nature, in the form of snow and rain, being negligible, as the water secured in this way was insignificant in quantity in comparison with that applied artificially. The average settlement of this portion of the embankment, in the 2 years following its completion, was less than $\frac{5}{8}$ in.

The writer believes that any one who observed the action of the moistened sand under the rollers would readily admit that they performed a very important part in the consolidation of the material.

The height of the ridges formed by the grooved roller was noticeably less with each successive passage of that machine; and the use of the two kinds of roller, passing alternately, was much more effective than that of either a smooth or grooved one alone would have been. Mr. Ferry.

The Sherardized bolts, referred to by Mr. Lynch, were lag-screws between the threads of which was spun a flat wire spiral to serve as reinforcement in transmitting the thrust from the screw to the concrete.

These bolts were mechanically defective in two respects. The wire spiral was likely to become slightly bent or sprung, in which case, only one thread of the screw would have a bearing, thus reducing the effective strength of the fastening. The defect which gave the most trouble, however, was caused by the machine which fashioned the bolt. This machine, in gripping the rod, changed the shape of the shank of the bolt, making the cross-section slightly oval instead of round, and, in addition, it formed a slight ridge along the sides of the shank along the major axis. The result was that, when the bolt was being unscrewed, it acted as a wedge which split a spall, several inches long, from the face of the step, thereby releasing the bolt. The only way in which this could be avoided was to give the bolt one or two turns just after the concrete had taken its initial set, thus forming a hole large enough to permit the bolt to be turned. If it was not turned until after the concrete had become hard, the head twisted off.

To repair the damage, dove-tailed recesses were chiseled into the steps, and the bolts were re-set in mortar. Several hundred bolts had to be re-set in this way before the writer succeeded in getting an order to discontinue their use.

In order to insure the head of the bolt having a firm bearing on the standards, the bolts were set about $\frac{1}{4}$ in. lower than the position they were expected to occupy finally. In spite of this precaution, however, the concrete broken off in unscrewing the bolts filled up the holes to such an extent that it was impossible, in many cases, to screw the bolts down again to a bearing, and it was necessary to drill out the holes and blow out the dust with a blast from an air pump, the threads of the screw interfering with the use of a spoon.

In the writer's opinion, whenever it is necessary to use screws for attaching fixtures to concrete it should be done by building the bolt into the concrete, rather than the reverse. If, however, it is desirable that the bolt shall be the movable piece, then the nut should be in an enclosing shell, thereby preventing the thread of the screw from coming into contact with the concrete. Several hundred bolts of this type were used, with entire satisfaction, for fastening the Holophane globes to the ceiling of the tunnels.

The difference in the movement of the two walls, to which Mr. Lynch calls attention, was undoubtedly due to a change in the design

Mr. Ferry. of the structure, rather than to any difference in the foundation, as that is of the same general character throughout the field.

The material at the field consisted of strata, of sand and gravel, of varying thicknesses. Among these were three or four layers of a very fine, compact sand—material which would be quicksand if in contact with water. One of these strata, about 6 in. thick, occurred about 3½ ft. below the base of the high retaining walls, and was uncovered in excavating for the foundation of the adjacent tunnels. The construction manager expressed grave fears that, unless the foundation of the wall was carried down through this stratum and built on the coarse sand beneath it, the wall would slip. As this stratum was about 20 ft. above the level of ground-water, and as no water could reach it from above after the embankment was concreted, there was no danger of the sand becoming "quick"; therefore the writer believed that, to support the wall, this material was equal to, if not better than, the coarse but less compact sand beneath it, and he strongly objected to the expenditure of the several hundred dollars necessary to extend the footing as proposed. In spite of his protest, however, the extra footing was built under one wall, the forms for the other wall having been erected and the steel placed in position before Mr. Atwood became connected with the work. This extra footing, under the outer edge of the base of the T-shaped wall, was built in the form of the frustum of a wedge, with the narrow edge down, thus reducing the effective bearing area of the sand about one-half, the sides of the wedge having comparatively little supporting value. This is the wall which Mr. Lynch refers to as having moved ¾ in. The companion wall, at the other end of the gate-house, which he says has moved less than ¼ in. is the same in all respects, except that it is 1 ft. higher, thereby sustaining that much additional pressure, but is built with a flat base, as planned originally.

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CONTROL OF THE COLORADO RIVER AS RELATED TO THE PROTECTION OF IMPERIAL VALLEY*

By J. C. Allison, Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. A. L. Sonderegger, J. A. Ockerson, and J. C. Allison.

Synopsis.

The general problems involved in the control of the Colorado River and the conservation of its waters for irrigation have already been treated exhaustively in papers and discussions before the Society, and it is not the intent of the writer to reopen the subject for purposes of debate. In this paper the direct relation of river control to the maintenance and ultimate perfection of levee systems protecting Imperial Valley, California, is the fundamental topic, as it has been the phase of the subject with which the writer necessarily has been most concerned.

In a broad sense, the problem of excluding the Colorado's overflow from the Valley has been solved. The maintenance of existing lines of defense, and the establishment of exterior auxiliary or supporting lines, may be considered as matters of detail. Final and complete protection, making possible the reclamation and beneficial use of all the arable lands of the delta, is to be secured, in the opinion which the writer holds in common with other engineers acquainted with the

* Presented at the meeting of September 6th, 1916.

situation, most satisfactorily by forcing the river back into the bed that it abandoned in 1910, and keeping it there with impregnable retaining works.

Methods of accomplishing that feat have been discussed, and estimates of cost have been made by several engineers, but methods and estimates have been based on emergency closures of breaks of the Colorado which, the writer believes, do not necessarily govern the case. Immediate protection being afforded by lines of defense which may be maintained with no difficulty and at small cost, the restoration of the river to its old channel is no longer an urgent need, and does not call for emergency methods. A favorable time may be chosen for the work, and methods less expensive than those heretofore considered may be applied.

It is safe to assume that engineers generally know the situation of Imperial Valley in relation to the Colorado, and only a brief statement of the problems presented to operators of the irrigation system will be necessary. The slope of the Valley from sea level at the Mexican boundary line is northward to the Salton Sea, 280 ft. below sea level. About 25 miles south of the line is the highest land in the delta, the elevation of which, before the change in the river's course, was 34 ft. above sea level at the westward end of the delta cone. The normal elevation of the river bed at the canal intake is about 100 ft. above sea level.

In flood season, which reaches the maximum in June, the river overflows the delta south of the crest of the delta cone, and fills a shallow sink at the west end known as Volcano Lake. The most rapid fall is southward, and the flood-waters find their way to the Gulf of California through Hardy's Colorado, which drains Volcano Lake; but the cone is so flat that some of the flood-waters normally spill from the lake northward into what was a shallow drainage channel known as the New River.

In 1905 the river broke through its bank north of the delta cone, or "Paredones" crest, and flooded the north slope of the delta. In draining off the inundated land to the Salton Sink, the water cut deep gorges in New River and in another drainage channel on the east side of the Valley, known as the Alamo. In the event of another incursion of the river, the gorges would carry the water, and there would be no inundation of farm lands on the American side. If per-

mitted to flow for any great length of time, the flood in the New River gorge would cut back the grade, and in time the recession would reach the old channel and lower the bed below the level of the intake, so that no water could be diverted into the main canal. Therefore the menace to Imperial Valley of an unchecked break would be drought, not inundation.

The problem presented by the change of course of the river through the delta in 1910, following the drainage channel known as Bee River to Volcano Lake, and thence through the Hardy, was the prevention of a northward flow into New River. That problem was solved by building the Volcano Lake levee, and by extending the Paredones levee of the California Development Company, parallel with the Bee River.

A deposit of silt raises the bed of Volcano Lake and consequently the surface elevation of the overflow water, and that has been met by increasing the height and cross-section of the levee. A small break in the levee in 1914 allowed some of the overflow water to go northward, but it was diverted easily from the head of New River gorge and turned into New River at a point where it did not produce any dangerous recession of grade.

Interior Lines of Defense.

North of the levee paralleling Bee River, which is now the main Colorado, interior lines of defense exist, and may be strengthened and extended very readily. The Inter-California Railroad grade serves as a barrier, and it may be connected with the Volcano levee. The function of interior lines would be to intercept and divert to the west a shallow body of overflow water having no defined channel and no eroding velocities. Numerous canal banks also serve as auxiliaries to check velocities and turn the water westward, where it may be taken care of readily.

The effect of one of these canal banks on the overflow of 1914 suggests a third line of defense and a method of diverting any water that might pass the Paredones and Volcano levees. This canal is the New Alamo Mocho, constructed by a rancher in Mexico to irrigate a large tract lying on both sides of New River channel. The water that came through the levee break, about 6 000 sec-ft., spread over the ground in a wide, shallow sheet, and was held by the canal bank, along which it flowed slowly to the west.

Assuming a breach in the Volcano Lake levee to be closed promptly, the inflow northward would be cut off before the slowly spreading sheet of water reached the west end of the proposed Alamo Mocho Canal bank, and it might not be necessary to let the water into New River at all. The overflow could be impounded and used as a reservoir for the West Side Main until drained off.

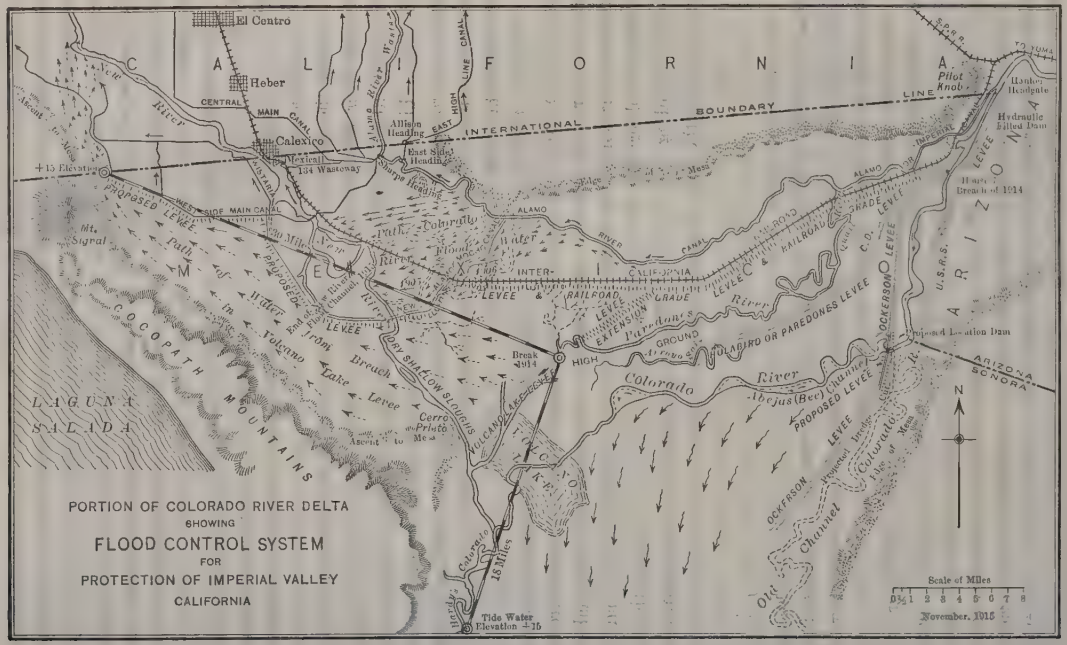
Each year prior to the break of 1905-06, which eroded the bed of New River, the overflow water from the Colorado entered on the northern slope of the divide. Each year this flow to the north subsided of its own accord, before the flow of the water to the south. The interior line of defense cuts off the water from entering New River gorge and so reinstates former conditions.

As an interior line of protection in emergency arising through possible failure of the primary lines, this proposed extension of the Alamo Mocho bank as a diverting levee seems entirely feasible and comparatively inexpensive to construct and maintain. It is important that the run-off channels leading southward from Volcano Lake be kept open and free from obstruction. Bee River has now cut a defined channel for itself through the silted Volcano Lake, and may be expected to keep to that approximate course for a long time.

Primary Line of Defense.

Only through negligence and failure to maintain the old California Development Company levee from Hanlon Heading south could the river again get into the region that was flooded in 1905-06, and be a menace to the Alamo Canal and delivery structures. The existing levee, properly maintained and protected from destruction by erosion of the banks of the river, is sufficient protection to the canal system as far as it extends. In 1914 a breach at the point where the levee bends to the southward resulted from negligence. Erosion of the river bank threatened the levee two years before the breach occurred.

The danger could have been averted by inexpensive means, but responsibility for the maintenance of the levee was a matter of controversy, and nothing was done until the bank and a point of the levee had been cut away by the river. Much money was wasted in an attempt to close the breach by the trestle and rock fill method on the line of the original location. The attempt was a complete failure, and the simple expedient of building a levee around the break was



PORTION OF COLORADO RIVER DELTA
SHOWING
FLOOD CONTROL SYSTEM
FOR
PROTECTION OF IMPERIAL VALLEY
CALIFORNIA

FIG. 1.

adopted, and the damage was readily repaired. In this case there was no danger of destructive flooding, as the river did not overflow the banks, and there was no head of water against the levee. The breach was caused by erosion of the bank and levee which could have been prevented by temporarily diverting the current away from the bank with jetties, until rock revetment at the toe of the levee could be placed.

The breach at "House 7", as the point is designated, was not due to any defect in the location or construction of the primary line of defense, but solely to failure to maintain the line. It was an administrative, not an engineering failure.

During the flood of 1914, the long stretch of bank between the levee and the river was cut away, up to the toe of the levee, but the latter was saved intact by dumping large quantities of rock at the foot of the slope, which filled the trench scoured by the current, and formed permanent revetment. A railroad track on the levee, and an inexhaustible supply of rock in a quarry at Hanlon Heading, near by, facilitated this method of revetment.

To make this line of defense impregnable, it is necessary only to reinforce the levee wherever bank cutting occurs, doing the work from year to year as conditions call for it. Assuming that reasonable attention is given to the matter, and that ordinary diligence in maintenance is observed, the protection afforded by the levee south of Hanlon Heading is sufficient. When thoroughly revetted down to the scour line, the levee cannot be broken by flood.

Head-Gate Foundation.

The only weak point in the line is the head-gate in the Alamo, or Imperial Canal, at Hanlon Heading; and the weakness there is due to the character of the rock foundation, which is not what it was assumed to be by the builders of the gate.

Some have held that the location of the gate in a rock spur of Pilot Knob was not necessarily of strategic importance, and that such a structure might be built safely and satisfactorily on alluvial soil, on the Mexican side of the line, using wooden caissons as a foundation.

Without going into the reasons for this proposed change of location of the head-works, which are political and, in the opinion of the writer, not based on engineering considerations, it seems almost self-

evident that rock foundation is a vital factor in the construction of a head-gate on the Colorado. This is demonstrated, not only by the failure of the structures lacking such foundations, but by the weakness developed in the foundation of the concrete gate at Hanlon Heading.

The Rockwood gate, constructed of timber in 1905, was built in the alluvial bank of the Colorado River in Mexico, some distance from the channel, to serve the double purpose of first controlling the runaway river, and later becoming the head-gate for the Alamo, or Imperial Canal. The writer is familiar with the design and construction, and all problems connected with that gate, and was on the ground at the very moment when it failed. It is his opinion that it would be difficult to devise, for a structure resting on alluvial soil, a more substantial foundation than was designed for this gate. In other words, the Rockwood gate represents a type of construction as nearly perfect as can be devised, at any reasonable expense, for a structure resting on alluvial silt deposits, and yet it failed.

In building structures, whether of wood or concrete, on alluvial soil, the problem presented to the engineer does not concern strength of material and design to meet the stresses to be put on it, so much as it involves provision to meet the boring action of water in front of and back of the structure, wherever large volumes are dealt with. For several months after it is built, the structure must be watched constantly because of the boring action and the resulting settlement or displacement of filling under and around the wings and curtains. After the settlement of the filling, the structure may be reasonably safe for the head under which it was "cured", but an increase or decrease of the head tends to develop new conditions which may cause damage.

This applies to Imperial Valley generally, but the difficulty is even more pronounced along the banks of the Colorado, where the land has been built up in strata of sand and silt containing more or less decayed vegetable matter, and where the structure is subject to extreme variations of pressure induced by a rapidly fluctuating river.

The Rockwood gate failed, not from the pressure of earth or water it was required to withstand, but from the boring which took place in front of the front curtain. In spite of the extended upper curtain, and the extensive apron built in front of the structure, this boring continued to develop with the increased volume of water put

through the gate, until it had entered under the front end of the structure and lifted it bodily from its anchorage.

This might have been avoided had rock and other material been available in quantities sufficient to prevent this scouring, but the same condition might have developed at any time under either wing of the structure, or from leakage through the silt puddle work when the structure was later suddenly subjected to increased water pressure. This is especially true because, during a recent seismic disturbance, the silt puddle work around most of the secondary structures in the canal system was disturbed.

The concrete gate at Hanlon Heading has been in use since November, 1906. During this time it has served its purpose as a regulator in the Imperial Canal, but with excessive operating costs, due to its design, and, as recently discovered, with considerable danger to the system during high water. The cost of handling drift through the gate has been excessive, and represents the largest operating expense of the river division, outside of the dredging of the canal. The enormous quantity of drift which the river brings into the intake and lodges against the gate has, in the past, blocked the openings to such an extent as to cause temporary water shortage in Imperial Valley.

As a partial solution of the drift problem, a sheer boom was built across the mouth of the intake during the season of 1912, and most of the surface drift has been handled successfully at that point, rather than at the gate itself. The submerged drift continued to give trouble by lodgment in the gate tunnels until a new section of the gate was built during the spring of 1913. The construction of the new gate was such as permitted the free passage of submerged drift. The level of the sill of the Hanlon Gate, as originally constructed, was calculated to pass all the water required for irrigation in Imperial Valley without a diversion weir in the river. The width of the structure was in excess of what was calculated for carrying water in the canal alone, as it was presumed that the gate would play a part in turning the water of the Colorado River back to its old channel. The sill proved to be sufficiently low to allow the desired quantity of water to pass over it into the Imperial Canal until 1912; but, because the average bottom of the river was lower during the low-water period than previously, owing to the new route which the river takes to the Gulf through the Bee River, and because the demands for

water during the low stage of the Colorado River were greater than was calculated, the sill of the gate proved to be too high.

During the summer of 1910 it became necessary to construct a rock weir across the Colorado River, in order to divert sufficient water through the gate to meet the demands. The necessity for the weir arose from the fact that the canal above and below the gate had silted, but since then it has developed that, even with the intake clear to the elevation of the former sill of the gate, sufficient water could not have been diverted at the low stage of the river.

It became evident that a second diverting weir would be necessary, or that the sill of the gate would have to be lowered. A permanent weir in the Colorado River was out of the question. The property of the California Development Company was in the hands of a receiver, and he could not do permanent work of this nature; neither were the funds in his hands sufficient to meet the costs of a permanent weir at this point. A temporary weir, such as was constructed in 1910, would not have served the purpose, on account of the heavy maintenance expense and the danger to the levees and banks on both sides of the river, caused by its presence during high water.

It was decided, therefore, to lower one 25-ft. section on the north end of the concrete gate, and in the new opening the radial gates were replaced with a sliding gate of the Stoney type. After the coffer-dams were in place and the excavation work had commenced, a flow of water developed around the end of the coffer-dam connecting with the original concrete gate, and thence inside the coffer-dam. A careful investigation disclosed the fact that the original gate did not rest on solid rock, but on a very inferior grade of decomposed granite, full of soft seams, all of which were water-bearing. This decomposed material had been scoured from under the floor of the old concrete gate until the piers and more than one-half of the original floor had scarcely any foundation left. Holes bored by the water under the original floor were in some places as deep as 8 ft. below the floor line. It is a marvel that the gate had stood, against a head of water varying in the summer from 13 to 16 ft., on the little remaining foundation, through which there must have been constantly a great quantity of percolation. Had this condition been allowed to remain through one, or perhaps two, additional flood seasons of the Colorado River, the remaining foundation would certainly have been

scoured out, causing the complete failure of the structure. This condition under the north side of the gate was corrected in that portion enclosed within the coffer-dam by a series of concrete curtain-walls running down in front and below the structure to serve as cut-off walls. The foundation was refilled with concrete, and a strongly reinforced concrete floor was laid on top. This section of the structure has since been used exclusively during high water, eliminating the danger of failure to a great extent. The weakest point in the primary line of defense, in the writer's opinion, is still the unused portion of the Hanlon gate. The weakness would be many times greater if the structure was in the alluvial deposits along the river bank in Mexico, where even an inferior grade of rock foundation is not to be had.

Temporary Diversion Weir.

Mention has been made of the temporary rock weir built across the Colorado River in the summer of 1910, and the statement has been made that such a weir would not serve as a permanent structure. H. T. Cory, M. Am. Soc. C. E., states that even the comparatively small quantity of rock dumped from the trestle to form this weir was not undermined, and did not settle, except to a slight extent and in a few places, with the summer floods of 1911 and 1912.

In building the weir, 1 822 cu. yd. of rock were used, at a cost of 60 cents per cu. yd. The total cost of the weir, including this rock, plus the cost of providing a trestle and maintaining it during its life, amounted to $32 350. The greatest head developed by the weir was 3.4 ft. on October 19th, 1910, when the Colorado River discharge was 5 000 sec-ft. The only way that this maximum head was developed and maintained was by constant expenditure of money and labor in obtaining rock and filling in the low places along the weir as they developed. In addition to this maintenance cost on the weir itself, there was the cost of maintaining the rock abutments on both ends of the weir up and down stream. The Arizona abutment was maintained at great expense some time after the weir was removed, in order to prevent cutting into the Arizona shore and injury to the levee constructed by the United States Reclamation Service. All this goes to show that such a weir is not practical, from a maintenance point of view alone.

In March, 1911, the trestle was removed and all maintenance work on the weir itself was stopped. For some time previous to March, 1911, no maintenance work was done because of instructions from the War Department that no more material should be dumped into the Colorado River. The average elevation of the crest of the weir was about 108 in October, 1910, and 105 in February, 1911, showing that as soon as the maintenance work was stopped, the rock began to slip down stream. In January, 1913, further investigation of the weir was made, and the elevation of the crest of the rock was found to be below 94, there being only a few places where the rock could be reached by sounding.

Very little good was effected by the weir in the way of securing sufficient discharge through the concrete gate at the low stage of water, after the summer of 1911. It is true that the piling butts, which remained after the blowing of the trestle, caught and held a great quantity of drift, and developed several sand bars in the location of the old weir, but this obstruction caused only a slight increase in the head at the intake.

Sole dependence for a sufficient water supply for Imperial Valley was placed on the hydraulic dredge *Imperial,* operating below the head-gates, and on the dredge *El Centro,* operating above the head-gate in the intake canal and in the Colorado River channel, controlling sand bar obstructions, etc.

The actual computed yardage of solid material moved by the dredge *Imperial* from the canal below the head-gate in one season's operations is equal to just one-tenth of 1% of the solid matter discharged through Hanlon head-gate in the same interval, but it is also a fact that it was just this one-tenth of 1% of solid matter which had heretofore blocked the canal delivery below the head-gate before the dredge was put in operation.

The major part of the one-tenth of 1% of solid matter removed by the *Imperial* was shingle, ranging from 1-in. rock to ¼-in. gravel, and sand ranging from gravel down to materials which are retained on a No. 10 mesh screen. It was the discovery of this fact that led to the successful operation of the Hanlon Heading, from the time of the entire removal of the rock weir in 1911 up to the present. This grade of material has been removed systematically every year since the dredge has been in operation, thus permitting the bed of

the canal below the head-gate to be maintained at an elevation which gave a sufficiently low water surface in the canal at all times. All other sands and silts not removed by the *Imperial* are carried down the canal in suspension.

The fact that there was deposited below the head-gate, at times when the velocities in the intake exceeded 6 ft. per sec., material of a kind which could not be carried down the canal by the velocities below the head-gate, which are never greater than 4 ft. per sec., led to the discovery of a means by which a hydraulic-fill dam could be placed across the Colorado River at a point directly below the intake, to serve as a means of diverting sufficient water into the Imperial Canal, should the necessity ever again arise. This time did arrive during the summer of 1915, because of the combination of an exceptionally low Colorado River during August and September, and an exceptionally great demand for water in Imperial Valley, due to the large increase in irrigated acreage over the previous years.

A study of the cross-section of the channel of the Colorado at Hanlon Heading, and at the gauging station of the United States Reclamation Service at Yuma (Fig. 2), shows that the Colorado—like other rivers forming deltas—scours and refills its channel bed with the increase and decrease of quantities of water, and that a direct relation exists between the character of the materials forming the bed and the velocities of the water. For example: The river bed is scoured to an elevation of 85 at Yuma, Ariz., by a discharge of 140 800 sec-ft., whereas it is refilled to an elevation of about 110 by a discharge of 3 400 sec-ft. The only reason that the bed does not scour deeper than an elevation of 85, with a discharge of 140 800 sec-ft., is that at that depth the materials constituting the bottom are so heavy that the velocities, registered as 6.8 ft. per sec. for this discharge, will not move them. It follows that, in refilling the bed, the materials are graded as to weight, and deposited in direct relation to the velocities. It also follows that the refill in all cases is in strata of materials decreasing in weight as they approach the elevation corresponding with the lesser velocities.

Though this is by no means a new discovery, the experience with the dredge *Imperial* below the head-gate served to bring out more strikingly the real relations between the weights of materials and

the velocities, and showed a way to make a practical use of this knowledge.

The scour line of the bed of the Colorado at Hanlon Heading is at an elevation of about 84. The ladder of the 10-in. suction dredge *El Centro,* operating in the river and above the concrete heading, was designed so as to make it possible to pump from an elevation of 80 when the surface of the river was 110. The materials in this area below the scour line had withstood the scouring effects of velocities as high as 7 and 8 ft. per sec. Those materials, when brought to the surface and deposited in the water where the velocity had decreased to 2 and 3 ft. per sec., due to falling discharge, were certain to stand in the position placed by the discharge of the dredge, making it

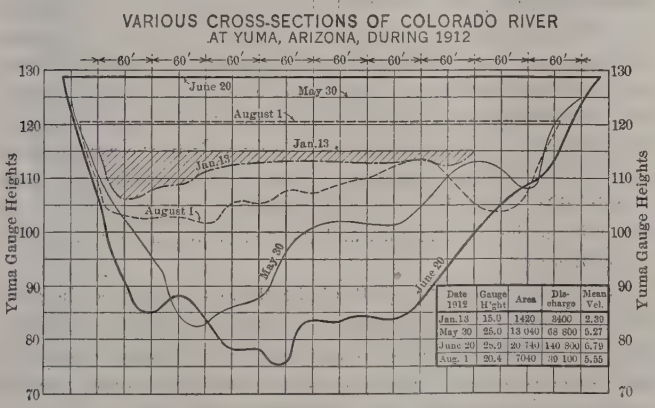

VARIOUS CROSS-SECTIONS OF COLORADO RIVER
AT YUMA, ARIZONA, DURING 1912
60′ 60′ 60′ 60′ 60′ 60′ 60′ 60′ 60′ 60′
130
120
110
100
90
80
70
Yuma Gauge Heights
June 20
May 30
August 1
Jan. 13
Jan. 13
August 1
May 30
June 20
Date 1912 | Gauge Height | Area | Discharge | Mean Vel.
Jan. 13 | 15.0 | 1420 | 3400 | 2.39
May 20 | 25.0 | 13 040 | 68 800 | 5.27
June 20 | 28.9 | 20 740 | 140 800 | 6.79
Aug. 1 | 20.4 | 7040 | 39 100 | 5.55

FIG. 2.

possible to construct a hydraulic-fill dam under the water of a running stream varying in depth from 2 to 15 ft.

HYDRAULIC-FILL DIVERSION DAM.

On August 12th, the *El Centro* commenced the construction of such a dam below the intake, and in 14 days had raised the fill to an elevation of 12 in. above the water surface three-fourths the distance across

the river. This fill reached from the Arizona bank of the river to within 250 ft. of the California shore, or a total length of fill of 650 ft. Due to the constriction of the channel during this period, a head of somewhat more than 1 ft. was developed, which resulted in a corresponding increase in velocity through the remaining opening. The velocities resulting from this head became as great as those developed during flood discharge of the river, and consequently material pumped from below the scour line could no longer be deposited in the dam, with any degree of economy, without adopting some method of decreasing the velocities. In closing the last 250 ft., therefore, a great quantity of brush and sacks filled with gravel was used.

The California shore had in years past been thoroughly revetted with a blanket of rock 25 ft. thick, making it safe to drive the channel directly against it, where the final closure was made. It is true that before the final closure took place the terrific velocities along the California shore scoured even the rock revetments to a certain extent, due to the large percentage of fine material composing the revetment, but a careful study of records, and the materials themselves, indicated the presence of a sufficient quantity of large rock in the revetment to make it a dependable protection against the cutting of the river on this side. The dam was commenced on the line of the old rock weir on the Arizona side, and the rock abutment served as protection there.

The closure of the final 75 ft. of the channel represented the most difficult operation in the construction of the dam, and the means devised for making this closure again demonstrated the advantage of close observation of the action of a river when left to itself, and the application of the results of this study in a practical way.

From August 12th to September 12th was the period of principal construction. The time subsequent to September 12th was spent principally in making the final closure and in maintaining the structure. On August 12th the river was discharging 8 800 sec-ft. Of this discharge, 2 529 sec-ft. were flowing into the Imperial Canal, all the gates in the heading being fully opened. The requirement for irrigation in Imperial Valley was 5 000 sec-ft. By September 12th, the river discharge had dropped to 4 000 sec-ft. During this period of 30 days there was used in the construction of the dam:

40 000 cu. yd. of sand and gravel fill, at 2.7 cents per yd.	$1 080.00
21 300 burlap and canvas sacks, at 3 to 6½ cents each	1 140.00
450 cords of willow brush and poles, at 73 cents per cord	328.50
800 lb. of wire	32.00
Cables and clamps	100.00
Labor, filling and placing sacks, carrying and placing brush, poles, etc.	2 000.00
	$4 680.50
Plus 10% for superintendence and tools, etc.	468.05
Total	$5 148.55

Since September 12th there has been an additional expense of $2 000 for raising and strengthening the dam by dredging, and for labor in closing and opening the dam, and general maintenance of the structure, making the total final cost $7 148.55.

The following methods were used in constructing the major part of the dam: Pumping was commenced on the Arizona side, the dredge pontoon being some distance above the site of the dam, and the discharge being carried astern through a 10-in. pipe floated on pontoons to the fill. Owing to the admixture of lighter grades of material encountered in burying the suction pipe to the scour line depth, and to the subsequent caving in on the suction of the surface materials when the depth was once reached, the base of the dam, in a great many places, became as wide as 150 ft. The heaviest materials pumped—in some instances stones and cemented adobe chunks 6 in. in diameter—remained exactly where placed, but the lighter sands and gravel were carried by the current some distance down stream before they lodged in permanent position. When a point near the water surface was reached with the fill, the conservation of the material was made possible by constructing brush fences to help retain the material at the slope required. (Fig. 3.) By this process the structure was carried across the stream to the point of final closure. Three re-runs across were made later with the dredge, bringing the crown to an elevation of 114, which is about 5 ft. above the water surface of the river when 5 000 sec-ft. are passing into the canal. (Figs. 4 and 5.)

On September 10th, 2 300 sec-ft. were passing through the opening in the dam, and 3 300 sec-ft. had been diverted into the Imperial

Canal. The head on the dam was 2 ft., and the resulting velocity through the remaining opening was 8 ft. per sec. The bottom of the opening had been paved with about 10 000 sacks, filled with gravel pumped by the dredge, and stoutly tied, decreasing the depth to about 5 ft., and thereby preventing scour. Great quantities of sacks were used in protecting the Arizona end of the opening, the California shore, as before stated, being protected by its rock facing. Additional sacks were used in forming the overpour riffles as fast as the head developed. A great number of the sacks were of no avail, either because of their lodgment in the wrong place, or because of breaking and washing out; and, throughout the whole process, constant replacements were required because of the boring effects at the toe of the sack riffles, causing the embedding of the sacks.

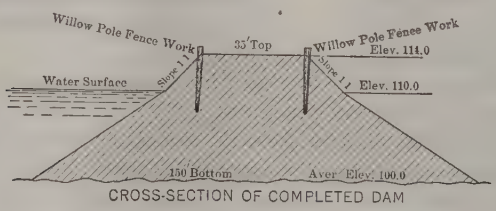


FIG. 3.

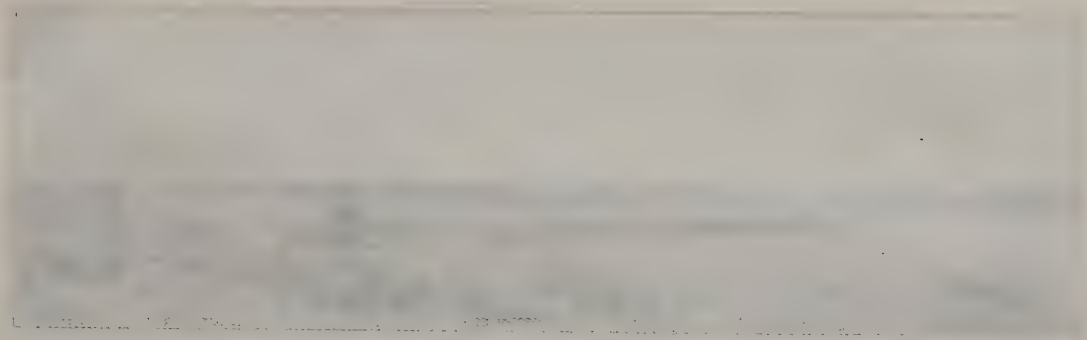
In paving the bottom, the point above the opening at which the sacks should be dropped in order that they might lodge on the bottom in the proper place, was determined by constantly experimenting with sacks thrown overboard attached to wires, thus making it possible to locate the sack when it had finally settled by following the line of the wire. Only those filled with the coarsest shingle and gravel and of greatest weight would remain in position and withstand the velocity for any length of time.

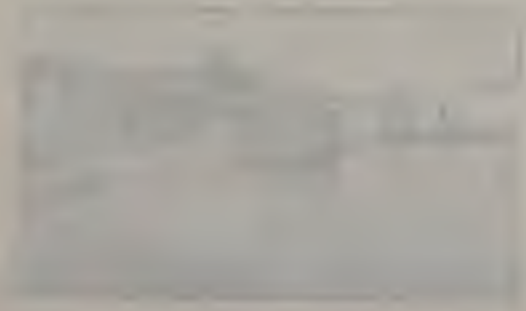
In the final closure, when the bottom had been protected carefully, cables were stretched across, securely anchored to deadmen on the California side and in the body of the dam on the Arizona side. The first cables used were of $\frac{3}{4}$-in. plow steel, but proved to be too light when maximum strains were put on them. After one failure in making the closure, due to breaking the cable, a double strand of $1\frac{1}{4}$-in. cable was used for the lower run.

FIG 4.—HYDRAULIC-FILL DAM IN COLORADO RIVER AT HANLON HEADING, FROM ABOVE ON CALIFORNIA SHORE, SEPTEMBER 11TH, 1915.

FIG. 5.—HYDRAULIC-FILL DAM, HANLON HEADING, FROM ARIZONA SHORE TOWARD ALAMO INTAKE CANAL, AUGUST 29TH, 1915.

FIG. 6.—CLOSING FINAL OPENING, HYDRAULIC-FILL DAM, HANLON HEADING, SEPTEMBER 11TH, 1915.





Willow poles were cut from the Arizona and California shores near by, where the supply is abundant. The poles were 30 ft. long, the diameter of the large end being 10 in., and the small end tapering to 5 or 6 in. By the skillful use of barges and lines working above the openings, these poles were maneuvered with the current in such a manner as to cause their large ends to lodge against the cable stretched across the opening above the water surface, while the small ends rested on the bottom at an angle of 30°, much as a drifting snag will lodge in a swiftly running stream when the heavy end is dragging the bottom and the upper end encounters a surface obstruction of trees or other large drift. (Figs. 6 and 7.) These poles were placed against the cable, in this manner, with enough space (sometimes 4 or 6 in. between them) to give sufficient strength to the structure and yet not decrease the area in the opening so greatly as to produce much additional head. When all the poles were in place, large bundles of willow brush were made up, weighted heavily with sacks, and rolled into the water from the barges, anchored some distance above the opening, in such manner as to cause them to strike the bottom of the channel and roll against the lower ends of the poles. As additional bundles were thrown in, the pressure against the poles increased, forcing them to bend, until in some instances, before final relief to the strain was obtained, the upper parts of the poles were standing vertically, and the lower sections were bowed to the extreme. Green poles were always chosen because of their flexibility. (Fig. 8.)

The placing of the bundles of brush was carried on in sections only, and only in sufficient quantity to decrease the velocity through the section to a point where material deposited by the dredge could be retained in place. In this manner the strain against the poles in any section of the opening was of short duration, as the dredge would give relief in a few hours by extending the dam above. Of course, a great many sections failed during the work, because of poles breaking, but means were at hand for speedy replacement when such accidents occurred. Because the impounded area above the dam was so great as to require several hours to increase its elevation when flow through the opening was reduced, the extra heads created by stoppage with the brush were not felt on the poles to any great extent before the dredge had given relief.

After numerous local failures, due to small freshet rises in the river, and the breaking of cables and poles, the opening was entirely closed on September 20th, the head then being 6.4 ft. (Figs. 9 and 10), and the discharge of the river, measured at Hanlon Heading, 2 958 sec-ft. Between September 20th and 27th the river discharge fluctuated between 3 000 and 4 000 sec-ft., all of it being required for irrigation in Imperial Valley. On September 27th, because of advance notice from Needles of a rise in the river, the dam was cut at a place provided, and all water, with the exception of 4 000 sec-ft. was passed through into the channel of the Colorado River below. The waters thus let through the dam were forced into the California bank below by local obstructions, so that on October 2d the opening in the dam was again closed, and all the water was diverted into the Imperial Canal. The discharge of the Colorado River on October 2d, as measured at Hanlon Heading, was 4 406 sec-ft. The second closing head was 8.2 ft. (The closing head of the Hind rock-filled dam, on November 4th, 1906, was 15.8 ft., with a river discharge of 9 275 sec-ft.*)

The 36 hours following the second closure were spent in clearing obstructions in the channel immediately below the dam, and in providing a new waterway to train the flow away from the California shore. On the evening of October 3d, the dam was again cut at the place provided, because of a rise in the river in excess of the water requirements in Imperial Valley. During the period from October 3d to the present writing, the opening in the dam, 100 ft. in width, has been left, and through it the discharge has been as great as 8 000 sec-ft. In constructing the dam, provision was made for accommodating the freshet discharges of the river, which commonly occur during October and November. These sections will be sacrificed only as required, because of the fact that during December and January the discharge of the river commonly reaches the minimum, and this being a year of exceptionally low water, it is quite likely that the irrigation requirements of Imperial Valley will again make it necessary to close the dam fully. Entire removal probably will be advisable in March or April, 1916, and by that time it will have fulfilled its purpose as a means of diverting sufficient water to meet the irrigation demands of the Valley. In any event, the annual flow period of May and June,

* "Irrigation and River Control in the Colorado River Delta," by H. T. Cory, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXVI, p. 1204.

FIG. 7.—HYDRAULIC-FILL DAM, HANLON HEADING, NEARING COMPLETION, SEPTEMBER 4TH, 1915.

FIG. 8.—REMAINING OPENING IN HYDRAULIC-FILL DAM, HANLON HEADING, SEPTEMBER 17TH, 1915.

FIG. 9.—FINAL CLOSURE, FROM BELOW HYDRAULIC-FILL DAM, HANLON HEADING, SEPTEMBER 20TH, 1915.

FIG. 10.—DREDGE, *El Centro*, PUMPING IN FINAL CLOSURE, HYDRAULIC-FILL DAM, SEPTEMBER 20TH, 1915.

1916, will remove the structure entirely. If the conditions which develop before this time should require it, the whole structure could be speedily removed by mining, as it is composed entirely of brush, sacks, and gravel, with the exception of a few willow poles driven with the aid of a jet from the dredge. The structure contains absolutely nothing of a permanent nature.

The experience in building this dam has developed naturally a great many improvements which would be adopted in a second construction of similar nature; so that such a dam could be built in a much shorter period and with much less expense. For instance, by constructing an aerial tramway, anchored to towers on each bank of the river, dredge lines can be carried across the stream during its flood, thus enabling the dredge to venture into the river during much higher water, resulting in earlier completion of the work. The ladder of the dredge *El Centro* is to be lengthened 15 ft., enabling it to pump material at a greater depth below the scour line. These two improvements alone will permit the commencement of the dam while the river is discharging 56 000 sec-ft., and will permit the complete diversion of a river carrying 15 000 sec-ft. if necessary. More economical use of sacks and the labor attending can be achieved by a more general use of cable and pole manipulation in closing the final opening. Fewer accidents in the final closure will be assured by anticipating the material requirements in time to accumulate sufficient quantities of poles and brush. By lengthening the pontoon and pipe line of the dredge, thus providing an increased radius of operation, a large supply of material suitable for construction can be obtained.

Closure of Bee River.

The application of a hydraulic-fill diversion dam as the fundamental factor in the closure of the Bee River break in the first line of defense is easily recognized. It is not possible to use the hydraulic fill to close the break itself and to force the river waters back into the old channel of the Colorado, because, in the vicinity of the Bee River break, the Colorado has never maintained a permanent channel bed for any length of time. Each flood, instead of scouring out a well-defined bed and refilling this with material suitable for dam construction, has widened and changed its channel to accommodate the larger volumes of water, deriving its increased sectional area from width, rather than

from depth. Very seldom is the river channel at the end of any flood season in exactly the same location as in the beginning, and for this reason there is no certainty that proper materials for constructing a hydraulic-fill dam at Bee River could be found. On the other hand, in the stretch of river channel from Yuma to a point 4 miles below Hanlon Heading, the river has always, within the memory of man, remained practically in the location it is in to-day, and the successful construction at any place within this radius is assured.

Engineers acquainted with the Colorado Delta agree that the final solution of the problem of control must be the restoration of the river to its old channel. Objections to that plan, which have been raised, are political, and do not apply to the soundness of the plan considered solely from an engineering standpoint. The inexpediency of doing any work of that magnitude in Mexico, until the rights and privileges of citizens of both countries in the matter of use of the river have been readjusted and defined, may be conceded.

There is no longer the imminent menace of disaster to the Imperial Valley canal system that made the closure of the Bee River breach in 1910 apparently an emergency job of vital importance. Construction and adequate maintenance of the line of levees from the old river bank to the west end of Volcano Lake, together with the provision of the interior lines of defense described in this paper, will safeguard the Valley for some time to come.

Therefore, the discussion of methods of closing Bee River and returning the Colorado to its channel close to the Sonora Mesa does not connote advocacy of immediate application of these methods. It is the writer's purpose to show how, in his opinion, such work may be done more readily and cheaply than similar work has been done heretofore, and that the costly processes proposed by some engineers are not essential to successful achievement of the end in view.

Diversion of virtually the entire low-stage flow of the river into the Imperial Valley canals, by the simple and cheap method herein described, removes the first difficulty otherwise to be met in making the closure. It is entirely feasible to make that diversion more nearly complete, and to turn a larger volume of water from the river into the intake and through the gates.

Having taken care of the low-stage flow and dried the river bed at the Bee River break, a levee or earth dam could be constructed

across the break with scrapers, in the same way as levees have been built along the river. It would be advisable, in the writer's opinion, to locate this dam nearer the old channel than the site of the dam with which the Government engineers attempted to close the break in 1910-11. The Government levee, from the C. D. levee down to where it was deflected westward near the break, is in fairly good condition, and should be repaired and utilized. A very important factor in the maintenance of the levee is a railroad on the crest connecting with the track on the C. D. levee. That is the vital factor in all levee protection on the Lower Colorado, a fact now recognized by the United States Reclamation Service engineers, who were unable to maintain the levee of the Yuma Project on the Arizona shore until they followed the example of the C. D. levee and placed on it a track connecting with the rock quarry at Yuma.

It is of primary importance that the Government levee be repaired and the track laid in time to extend it over the Bee River dam as soon as the dam has been completed by the scrapers, in order that rock from the Pilot Knob quarry may be used immediately for the revetment of the levee and dam.

Changes in the location of the dam, and of the connecting levee for short distances from both ends, seem advisable. An important reason for the changes is the resulting reduction of head against the dam and levees during overflow from the old channel.

If the work was to be done as originally projected, doubtless a better location than that of the Government levee could be made, nearer the Sonora Mesa, for the reason that the less room the river is allowed for meandering, the greater its velocity and scouring action, and the easier it is to keep it under control. The advantages, in the case of the Colorado River, of a narrow, confined bed, are to be seen plainly in the stretch of river from Yuma down to a point 4 miles below Hanlon Heading, and, in the writer's opinion, they completely refute the theory that the river should be permitted to meander erratically through the delta. The nearer the river course can be made to an efficient hydraulic channel, the simpler becomes the problem of control.

However, the advantages of a new location may be considered as more than offset by the advantages of the utilization of work already done, one of which is the settling, consolidation, and "seasoning" of

the sections of the Government levee remaining. The original disadvantage of river side borrow-pits has been minimized to a great extent by the growth of brush and willows that has choked them. As time elapses, that obstruction to the flow of water along the foot of the levee increases. It is essential that, in the restoration of the levee south of the break, the river face shall be blanketed with gravel and the track extended to the south end.

The old channel, from the Bee River diversion some 5 or 6 miles southward, would have to be cleared, and it would be advisable to shorten the course by cutting across some of the bends, thereby augmenting the velocity and the scouring action. In opening the old channel, it would be advisable to use a dipper-dredge, making a double cut for about 6 miles. The writer made a reconnaissance of that region recently, and found outside of the old channel a thick jungle of brush, but no big growth and very few trees. The work of clearing off the lines ahead of a dredge would be inconsiderable, as the dredge could handle the brush. In the old river bed, however, the growth of brush and trees has been very rapid and heavy, and it would be necessary to do thorough clearing, and perhaps it would be advisable to break up root growth and loosen the ground with dynamite.

Given a good, clear start down the old channel, the river in flood would cut an adequate course for itself.

Having a track on the levee from Hanlon Heading to the south end, and the quarry at Pilot Knob to furnish unlimited quantities of rock, loaded directly by steam shovel, revetment could be placed readily wherever danger of bank-cutting under the levee might develop. It would not be necessary to revet the entire levee at once. That work should be done as needed, as part of the maintenance, and spread over a long term of years.

Conclusions.

In discussing the control of the Colorado River, as related to the protection of Imperial Valley, most space has been devoted to describing the hydraulic-fill dam, recently constructed across the Colorado at Hanlon Heading, and the possibility of its further use in future river control.

The writer does not mean to state that such a structure could have been used in stopping the breaks of 1905-06, because of the emergency

nature of such work. It is not presumed that such methods can be applied in cases similar to those of the past, where the breaks were stopped while the Colorado River was practically in flood, and where the resulting closing heads were so great as to prohibit the adoption of such means as were used in the recent dam.

Necessity for further emergency work to safeguard Imperial Valley from the flood-waters of the Colorado River no longer exists, and therefore it is possible in the future for the engineer to choose his construction period to conform with the low stages of the river. All the hydrographs of the Colorado River indicate a low period during August, September, December, and January of each year, with very few exceptions, and surely the work can be planned in anticipation of these low stages.

If the engineer will look about him and apply the means at hand in his work, and will study the characteristics of the stream more closely, no such expenditures as have been made in the past will ever again be necessary. The hydraulic-fill dam supplants the trestle and rock-fill dam previously used, which, of course, represents the greatest item of expense in the closure. Not only here is opportunity for great saving of both time and money, but in almost every item of construction is the same opportunity, if the engineer will profit by past experiences and study more closely the examples Nature has provided for him.

DISCUSSION

Mr. Sonderegger.

A. L. SONDEREGGER,* M. AM. SOC. C. E. (by letter).—There are three engineering problems which to-day confront the Imperial Valley Irrigation District and are more or less related to each other: First, the protection from overflow of the Colorado River; second, the silt problem; third, the problem of diverting the low-water flow of the river during the latter part of the summer, when the greater portion, or all, of the flow of the Colorado River is required for irrigation in the valley.

Mr. Allison presents a simple and practicable solution of the first problem. The efficiency of the proposed system of defense can hardly be questioned, as it has already had its practical test, at least in part. The plan includes three main lines of defense.

First line: The diversion of the Colorado River into its old channel along the Arizona and Sonora Mesa; the closure of the breach of the Ockerson levee at the head of the Bee River, together with the restoration of the Ockerson levee, or a new levee similarly situated.

Second line: The westerly portion of the C. D. levee and its prolongation, called the Paredones or Holabird levee, and to the west thereof the Volcano Lake levee.

Third line: The embankment of the Inter-California Railroad as far west as the Alamo-Mocho Canal, and a new proposed levee along this canal and westerly thereof to the West Side Main Canal just south of Calexico.

Of the first line of defense, the upper part of the old C. D. levee, from Hanlon Heading to its junction with the Ockerson levee, has withstood successfully the undermining of the river, wherever riprapped with rock. The break in the levee at "House 7", which occurred in 1914, was due to lack of maintenance. Nevertheless, this first section of levee forms the weakest portion of the whole system of defense.

Mr. Allison has referred to the undermining of the foundations of the old concrete piers built on the granite ledge at the Hanlon head-gate. The writer is quite familiar with these foundations, as the new Stoney gate was put in, during the winter of 1913-14, under his direction as Consulting Engineer for Mr. W. H. Holabird as Receiver. It developed that the granite ledge was a shattered mass of soft rock with many seams of disintegrated granite and strata resembling talcum, sometimes several feet in thickness. There was no need of any blasting, all excavation being made with pick and shovel. The foundations uncovered under the old piers within the temporary coffer-dam were badly undermined. In one instance there

* Los Angeles, Cal.

Mr. Sonderegger.

was a cavity under the down-stream side of a pier large enough to contain an office desk. The 8-in. concrete floor-slab was broken off and eroded in many places. Soundings along the remaining piers showed that they also were badly undermined, although calculations regarding their stability revealed a small margin of safety. Lack of funds prevented the Receiver from repairing the foundations of these piers, and they still constitute a danger which should be eliminated. A granite quarry is within 1 000 ft. of the heading, and, in case of a break, rock could be dumped without delay.

From the heading down to the junction of the Ockerson levee there is only a single line of defense. This section of the levee has already been revetted for some distance. If the levee is watched, no danger need develop, as it carries a standard track, and rock can be dumped at short notice wherever the current approaches the levee. Some of the revetments were lost in January, 1916, owing to absence of maintenance.

Regarding the location of the Ockerson levee, the writer agrees that, as far as river control is concerned, it is of vital importance to concentrate the flow of the river, and to prevent meandering between levees.

As regards the proposed method of closure of the Bee River by a temporary diversion of the Colorado River at Hanlon Heading during the low-water stage, two facts seem to stand out very clearly: first, that the construction of a hydraulic-fill dam can be accomplished successfully; and second, that it can be done at very low expense.

The difficult part of this work—the closure of the last gap in the dam—was effected successfully, not only once, but several times, when it became necessary to cut the dam for the passage of temporary floods; therefore, there can be no doubt as to the success of any future operation of this kind. Both the method and the low cost bear witness to the ingenuity and resourcefulness of the directing engineer.

Considerable dredging will be necessary in the old channel below the Bee River Heading before the river can be diverted permanently. It is probable that the work will extend over a period of 2 or 3 years before a channel can be secured capable of accommodating the winter flow and occasional freshets as they occur; but, unless there are high waters of an extreme nature, and out of season, the writer believes that the diversion can be effected without any great risk.

The second line of defense—the C. D. levee, together with the Holabird and Volcano Lake levees—had proved its efficiency for several years, until in 1914, in the absence of maintenance, a break occurred. The difficulties encountered in the attempt to close the breach indicated clearly the necessity of establishing permanent connection by rail to the Hanlon Quarry. This was done in 1916. This Volcano Lake

Mr. Sonderegger.

levee to-day presents the only protection for the valley. It is partly revetted with rock.

The third line of defense, especially the proposed levee along the Alamo-Mocho Canal, and westward therefrom, also had a partial test during the break in the Volcano Lake levee of 1914. At that time the bank of the Alamo-Mocho Canal intercepted the floods issuing from Volcano Lake, causing them to spread out in a thin sheet over the country to the south and west thereof. This canal crosses the New River channel on a solid embankment, and although it ends about 5 miles to the west thereof, it effectively prevented a concentration of flood waters for several days. The water finally found its way around the end of the canal bank into New River. The levee proposed by Mr. Allison would perfect the existing system of protection.

During the summer of 1916 a new canal was built from the west end of Volcano Lake levee, in a northerly and northwesterly direction, to the intersection of the Wistaria Canal with the West Side Main Canal. This point is about 3½ miles south of Calexico. This canal forms the first side of a triangle, of which the Volcano Lake levee and its extension to the Inter-California Railroad forms the second side, and the railroad levee and the Alamo-Mocho Canal, with its proposed westerly prolongation, the third side. The area over which a flood from Volcano Lake might be spread has been reduced by the construction of this new canal, and a deflection of flood waters to the west is made impossible, unless the new canal is cut. The lowest point of the triangle is at Wistaria Heading. The levee proposed by Mr. Allison, and running from the Alamo-Mocho Canal westward, is still a necessity, in order to close the third side of this triangle and thereby make the third line of defense fully effective.

The question naturally arises as to the effect, on the grade of the river, of the proposed diversion of the Colorado into its old channel: Eventually, it would affect both the levee protection work of the Yuma Project and the diversion into the Alamo Canal of the low-water flow of the river.

From the reports of Col. Ockerson and Mr. Randolph it must be inferred that in 1911 the slope from the head of the Bee River to Volcano Lake was about 2.2 ft. per mile, while along the old channel of the river it was 1.08 ft. per mile. Since 1911 enormous deposits of silt have been carried into the depression of Volcano Lake, so that the grade to the lake has probably decreased. However, there must still be a preponderance of grade in that direction.

It is a fact that the diversion into the lake did cause a recession of grade extending at least as far up as Yuma, and this fact has undoubtedly contributed to the difficulty of diverting the low-water flow into the Alamo Canal. The trouble has become very acute, not only in

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the case of extreme low waters, as in 1915 when the river fell to a discharge of 3 000 sec-ft., but also with moderate volumes, as in 1916, when a water shortage occurred in the valley, with a low-water average of 25 000 sec-ft.

It is to be expected that the restoration of the old channel will cause a raising of the river bed, especially at low-water stages. On the other hand, the confining of the flow between the Arizona Mesa and the levee on the west side may cause a concentration of flow at high-water stages, and, simultaneously therewith, a scouring.

Since low waters occur during August, the question of securing a sufficient supply for the valley is second in importance only to the question of defense. It is, furthermore, intimately related to the silt problem, and the two questions will be discussed together.

While in the employ of the Receiver of the California Development Company, the writer, in conjunction with Mr. Allison, made extensive studies regarding the phenomena which affect the diversion of the river at Hanlon Heading.

It should be remembered that the granite ledge which carries Hanlon Heading, is not on the banks of the river, but approximately 2 000 ft. below it. The sills of the old gates are at an elevation of 98 ft., and of the new Stoney gate, at 93; and the low-water stage of the river, of late years, has never fallen below an elevation of 104.3 ft. Theoretically, there has been sufficient fall to effect a diversion, especially since the erection of the Stoney gate in 1914. At the intake of the canal, water is diverted at an angle of 90°, which naturally causes a decrease in velocity from that along the axis of the river. The result is the formation of a bar at the intake.

During high waters this first section of 2 000 ft. of canal above the gate operates as a settling basin. It is about 100 ft. wide, and varies in depth from 3 to 15 ft. With a rising river, the bottom of the canal rises practically at the same time as the water level of the river, causing a silting up of the canal to a depth of 10 ft. or more. Apparently, the cross-section of the canal and the velocity of the water are regulated automatically, corresponding to the volume diverted and the quantity of silt carried by the river. The swift currents carry heavy silt, shingle, and driftwood, large quantities of which are diverted into the canal, having a tendency to pile up in front of the head-gate. Desirable as it might be to prevent the admission of these silt-laden waters, yet the present position of the head-gate makes it necessary to keep its sill clear, so as to prevent the formation of mud banks and log jams immediately above the sill. Once such a bank is formed, it cannot be flushed out, and, with a sudden drop in the river, the heading might be left high and dry. The practice adopted by English engineers with Indian canals—providing gates with two or three leaves,

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which enable them, during high water, to skim the water off the top—cannot be adopted for the Alamo heading with the head-gate in its present location.

Immediately below the heading the canal enlarges to a bowl shape with a width of about 200 ft. Here the waters, rushing through the gate, are stilled, and drop their heaviest detritus, the effect being that a bar is formed a few hundred feet below the heading. The width of this bar varies from 1 000 to 2 000 ft. During the high water of 1914, this bar reached an elevation of from 102 to 103 ft.

High waters generally occur in June and July, after which the river drops suddenly. Experience has shown that, if the sill is kept clear, the section of the canal above the gate is scoured practically simultaneously with the drop of the water level of the river, leaving, however, a bar at the entrance. To dispose of this the 10-in. suction dredge *El Centro* is used, and no difficulty has been experienced in keeping the channel open.

Below the gate, however, the bar is not scoured out by a falling river; in such a case, the grade of the intake canal is governed by the water surface in the river, and the elevation of this bar is independent of the elevation of the sills of the gate. In fact, the maximum elevation of the bar has been from 4 to 5 ft. higher than the sills of the old miter gates, and from 9 to 10 ft. higher than the sills of the Stoney gate. The only means of eliminating this bar has been the operation of the 15-in. suction-dredge *Imperial*. In 1914 the bar was reduced to an elevation of 99 ft., sufficient to divert the necessary low-water flow; but in 1915 and 1916 it has required the raising of the water level in the river by temporary dams.

The situation, therefore, is as follows: During high waters large quantities of heavy silt and shingle enter the canal, forming a bar below the head-gate, the elevation of which depends mainly on the high-water conditions of the river. With a drop in the river, its velocities decrease, and the volume of silt carried is reduced correspondingly. The canal then begins to scour in the section above the gate, lowering the bottom automatically. If the drop in the river is not too sudden, this scouring keeps pace with the lowering of the water surface, however, leaving a bar at the intake; but, below the gate, apparently no scouring action occurs, and it requires the operation of a dredge to reduce the lower bar, or the construction of a dam to raise the water level in the river.

In view of these facts, the immediate effect of the restoration of the old channel of the river on the elevation of the low-water level at Hanlon, and the problem of diverting the low-water flow, cannot be predicted with certainty. Undoubtedly, in time, it would be beneficial; but how soon the effect would be noticeable, and whether it

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would be sufficient to eliminate the present difficulty completely, are still open questions. In solving the problem, the most important feature is to prevent the heavy silt, which rolls along the river bottom, from entering the canal; especially at high-water stages, only the surface waters of the river should be admitted.

This can be accomplished by constructing head-works along the banks of the river at a point where the current will strike the gate permanently and keep the sill clear.

It is generally admitted that the construction of a permanent dam near Hanlon Heading is excluded, for many obvious reasons. The writer agrees with Mr. Allison that it is not advisable to construct a new heading on the banks of the river on an alluvial silt foundation, even if provided with a rock-fill foundation and extensive rock-fill aprons. This would probably be the method of construction to be adopted if a gate were placed, as has been suggested, at the site of the well-known break of 1906 which was closed by a rock-fill embankment. It is difficult, if not impossible, to estimate the extent of the boring action which will take place in front of a gate during an extreme high water. Undoubtedly, the action is quite different from that occurring along a revetted bank. Provision would have to be made for the maintenance of the rock fill, by a system of tracks commanding the entire up-stream apron and its wings. In order to avoid excessive vibration and a slow wreckage from such forces, an extremely massive concrete floor under the gate proper would have to be provided. At least, a gate in such a location would require unceasing vigil, and present a constant menace to the valley.

However, a location which is worthy of investigation exists on the west bank about 1½ miles above Hanlon Heading, and the writer is informed that this locality is now under consideration by the Imperial Irrigation District.

The bank at that point is on the convex side of the river, and is swept by the main current during the greater part of the year. It consists of a ledge of conglomerate which apparently presents some resistance to the boring and scouring action of the river. It will require careful soundings and investigation before its suitability can be determined definitely.

If the result should be found to be satisfactory, a heading can be constructed in a manner that will prevent heavy silt from entering the canal. The writer believes that even in the absence of special settling basins, the quantity of silt admitted to the canal can be reduced materially, and that this would produce the effect of scouring in the Alamo Canal. The present grade of this canal is practically stationary, and is the result of the discharge of waters carrying extremely high percentages of silt and sand. If operated under the

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same grade, but with comparatively clear water, a scouring action is sure to occur. It is a phenomenon often observed in the canals in Imperial Valley.

Probably, as far as the average low-water discharge is concerned, there would be no difficulty in diverting the quantity necessary for irrigation; for extreme low waters, as they have occurred in 1912, 1913, 1914, and 1915, when at times practically the entire flow was diverted, there would still be the necessity for the construction of temporary dams—whether they are hydraulic-fill or rock-fill. It should be remembered that increasing quantities of water will be diverted for the Yuma Project as all its lands are brought under cultivation. Similarly, there is still a constant increase in the acreage in Imperial Valley, so that within a few years the entire low-water flow of the river will be required. This is exemplified in the report* by E. C. LaRue, Assoc. M. Am. Soc. C. E.

It may be stated, therefore, that the reconstruction of the heading on the banks of the river, 1½ miles above Hanlon, even if found practicable, will present but a partial solution of the problem of diverting the low-water flow, and, likewise, only a partial solution as regards the elimination of silt, provided the river maintains its present channel. The successful removal of all silt by settling basins—in the opinion of the writer—depends entirely on the operation of the proposed new heading.

Another plan for securing a sufficient supply of clear water for Imperial Valley is the proposed construction of a canal from Hanlon Heading to Laguna Dam. There will be no engineering difficulties in the construction of the connecting canal. The proposed route crosses several large washes which, at long intervals, are subject to sudden floods resulting from cloudbursts. However, there is sufficient grade available, so that short portions of the canal can be constructed as underground conduits with heavy grades and correspondingly reduced cross-sections; thus, the cost of construction is not likely to be excessive.

There is one feature of this plan which will require careful consideration, namely, the necessity for the continuous operation of the Imperial Canals. The Laguna Project at present diverts probably not in excess of 750 sec-ft. during the summer. In order to have the benefit of the settling basin above the dam, it is necessary to flush the settling pond immediately above the intake one day per week, so that the canals are operated 6 days out of the 7. The distance from Laguna Dam to Yuma is about 12 miles.

It is doubtful whether this mode of operation could be applied to the Imperial system. The summer diversions would be increased from 750 to about 6 000 sec-ft., and, during the low stages of the

* *Water-Supply Paper No. 395.*

river, every drop would be diverted. The distance from Laguna Dam to Imperial Valley is about 75 miles, and, in view of this, it is doubtful whether intermittent operation would be successful. In other words, though the connection with the Laguna Dam would undoubtedly secure a permanent supply, it would not solve the silt problem completely. Mr. Sonderegger.

Whatever solution will finally be chosen, it will necessitate the expenditure of large sums of money, and on the success of the undertaking will depend the future of Imperial Valley. No decision should be made, therefore, except after full and mature consideration of all questions involved.

J. A. Ockerson,* Past-President, Am. Soc. C. E. (by letter).—In this paper, Mr. Allison presents some very interesting and novel features. His long experience in connection with the irrigation of Imperial Valley gives exceptional weight to his views as to the proper solution of the vexing problems involved. Mr. Ockerson.

Complicated by International Boundary Line.—A comprehensive plan for the reclamation of the Colorado River delta is complicated by the presence of the International Boundary Line between the United States and Mexico.

If that was eliminated, the restoration of the river to its former bed and the limitation of its floods thereto by suitable levees or embankments, together with such revetment as might be necessary to check and prevent excessive bank erosion, would solve the problem of regulation definitely, so far as the river itself is concerned. "Interior lines of defense", which, at best, are temporary makeshifts, would then be unnecessary.

The water of the river, however, is necessary to supply moisture to about 500 000 acres in Imperial Valley, California, and about double that area on the Mexican side of the line. This is more than double the total area of land reclaimed by the United States Reclamation Service, and Imperial Valley alone has a cultivated area measuring more than two-thirds of the total of all the Reclamation projects. This gives a fair measure of the importance of the projects under discussion.

It is the diversion, control, and distribution of this water in Imperial Valley which has largely claimed Mr. Allison's attention for several years, the development of the Mexican side being comparatively small.

The New River Gorge.—The development of the New River gorge is commonly attributed to the break of 1905, when the Colorado River left its channel and emptied its waters into Salton Sea. An investigation of the earliest records left by the stream itself in its wander-

* St. Louis, Mo.

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ings shows plainly that this diversion to the Salton Sea had occurred many times in the distant past.

Early Mexican maps, dating back to a time before any attempt had been made to divert and control the waters, show the New River gorge to have been developed to a considerable extent even then. The rapid enlargement of the gorge during 1905-06 was most impressive as an example of excavation on a gigantic scale, such as would make the Panama Canal work seem insignificant, and has rarely been equalled elsewhere in Nature.*

Break in the Levee at "House 7".—The author speaks of the breach in the levee at "House 7", and suggests that the use of "jetties" to deflect the current away from the bank would have prevented the damage until rock revetment could have been applied. He also states that there was "no danger of destructive flooding, as the river did not overflow the banks". If the records of the Reclamation Service are correct, the river banks at the point named have never been overflowed by the highest known floods, and why "the simple expedient of building a levee around the break", or why a levee 9 ft. high was ever built at all on a bank which is 4 ft. above the highest known flood, has always been a mystery to the writer. It is fair to say that the author was not responsible for either of these.

Deflecting Dikes or Jetties.—The use of dikes or jetties to deflect the current away from the banks is a common delusion. R. Jasmund, in his review of the regulation of the Rhine, covering some 150 years, recites the "countless failures" in which deflecting dikes had a very large share. He says that, in spite of such dikes inclining down stream, the bank receded more and more and, in many cases, the erosion was strongest just behind the dikes, where protection had been looked for, and the dikes themselves were cut out and disappeared. After being thoroughly tried out, the deflecting type of dike was abandoned, and dikes of suitable length perpendicular to the bank were substituted, which resulted in deposits along the shore below them, which method has had a large part in the successful regulation of the Rhine and other streams.

Like experience on the Mississippi and Missouri Rivers has also demonstrated the futility of current deflectors in the protection of river banks from erosion.

Levees Along the West Bank.—When the break of 1905 was closed, a levee was built to the southward for a few miles, and then turned westward toward the old overflow channel known as the Paredones. It was well known at that time that a break in the near future was imminent by way of Bee River. If, instead of carrying the levee back away from the river, it had been extended down along the river, across and well below the Bee River, which was dry except at flood

* *Transactions*, Am. Soc. C. E., Vol. LIX, Plates V and VI, and p. 37.

stages, the break of 1910 with its attendant losses never would have occurred. It would also have been in line with "final and complete" regulation as advocated by "engineers best acquainted with the problem". Mr. Ockerson.

The writer discussed this matter at the time with those in charge of closing the break, and understood that the plan was to extend the levee down along the west bank of the river to what was reported to be high land well down toward the lower end of the delta. There are, in fact, evidences on the ground, in the nature of a partly constructed levee for a short distance below the point where the levee turns to the westward. It would be interesting to know what caused the change in the plans.

Hydraulic-Fill Dam Across a Flowing River.—The author's success in building, across a flowing river, a hydraulic-fill dam out of material pumped from the lower strata of the bed of the stream is a striking example of what a resourceful engineer can accomplish when an emergency arises.

In his official report of 1910 the writer suggested that the whole operation of the closure of the Bee River break "might be economically handled in the dry", by operating during a low-water season, using the temporary rock-fill dam built just below the intake, and raising it to the required height to divert the whole flow of the river through the Imperial Canal.

The writer's experience does not agree with the statement that the depth of scour is limited by the heavier material composing the bottom, and that "the materials are graded as to weight, and deposited in direct relation to the velocities". In the Mississippi River, the bars composed of gravel larger than the shingle described by the author, extend well above low water, and the deep concave bends opposite the bars have no gravel in them at all. It is not uncommon to see coarse gravel distributed over the high-water banks. Take the exposed section of an alluvial bank in general, all of which has been deposited from a state of suspension, and there is little evidence to show that the materials are graded as to weight.

The scour line of the bed of the river at Yuma is also doubtless influenced by the bed-rock which extends across the river in that vicinity.

The construction of the dam described was only possible during the extremely low stage in the river, amounting to only 4 000 sec-ft., all of which could easily be carried through the head-gates to Imperial Canal. The low cost of the work fully justified the effort, and its novelty and successful outcome reflect great credit on the builder.

It is stated in the press that there is under construction a new temporary rock-fill dam which will cost several times as much as the

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hydraulic-fill dam under discussion, and it would be interesting to know why the latter was not repeated.

Restoration of Colorado River to Its Former Bed.—The restoration of the Colorado River to its former bed must be realized before a final solution of the problem of control is reached; and, when it is undertaken, the experience gained in the construction of the hydraulic-fill dam can be applied with success. As the author well says, closure should not be attempted under emergency conditions, as heretofore, but a favorable time should be selected, when the whole volume can be diverted down the canal through the head-gates, and then it will be possible to build "dry-shod" such dam or barrier as may be necessary to turn the flow back to its old channel. The latter would also probably need clearing out and rectifying, in order to facilitate the flow.

Railroad on Crown of Levee.—The writer cannot agree that a railroad on the crown of the levee is a "vital factor in all levee protection on the Lower Colorado"; but it may be a convenience in the work of maintenance, which cannot be neglected if the levee is to remain intact.

The writer is of the opinion, also, that, in the use of rock to protect the exposed face of the levee from erosion, economy as well as safety dictates that the rock facing shall be laid by hand with some care, instead of being dumped at random from cars, as a large part of the rock thus dumped serves no useful purpose; witness the levee north of Bee River, which was breached in spite of such protection.

Restricting the River to Narrow Limits.—The author states that "the less room the river is allowed for meandering, the greater its velocity and scouring action, and the easier it is to keep it under control". This statement needs qualification. When the Sonora Mesa is approached, only one bank of the river requires control by revetment; the Mesa itself takes care of the other bank, as do the hills on the right bank from near Yuma to Andrade, which the author cites as a narrow confined bed. Increase of velocity necessarily increases bank erosion, as well as the difficulty of control by bank revetment. Increased velocity does not mean that the energy will all be expended on the bed of the stream, for it will also attack the banks. Therefore, why not give it more lateral room, so as to reduce the flood height?

Location of the Government Levee.—The location of the Government levee on the west bank of the river was fixed after a careful study of the changes in the bank line, as shown on a map defining several positions of the river dating from the earliest surveys.

This study developed the fact that, had a levee been built 20 years earlier, and placed at a distance of 3 000 ft. from the westerly bends of the river, it would have remained intact, as far as river bank erosion was concerned.

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As the project of 1910 could not cover bank revetment, it was decided to place the Government levee so that it would be safe from destruction by river bank erosion for a probable period of about 20 years, the primary object of the levee being to carry the flood-water so far down stream as to eliminate the possibility of its flowing into Imperial Valley. With the exception of the detour to connect with the Bee River crossing, which was regarded as the best available point, the levee alignment was fixed in accordance with this plan.

In a comprehensive scheme for the regulation of the Colorado River, including bank revetment and flood control, the levee alignment might be somewhat different.

River Side Borrow-Pits.—The author seems to object to river side borrow-pits, which constitute a cardinal principle, followed in all parts of the world, except in the Colorado River delta; and in most of the levees built there that practice has been followed, except when prevented by the encroachment of the water on the river side.

An examination of the Government levee after the floods of 1911 disclosed the fact that there were no channels in the pits paralleling the levee, but, on the contrary, the pits in many cases were obliterated by deposits of sediment. There was no evidence anywhere that the river side pits were responsible for damages to the levee.

The imperative necessity for at least temporary protection of the river slope of the levee was fully realized by the writer, but, when proposed to those "higher up", was rejected, because there was "no authority under the appropriation, or negotiations with Mexico for maintenance".

Hanlon Head-Gates.—Apparently, very little attention had been given to the hydraulics of the river, its variations in stage, and the oscillation in the elevation of the bed conforming in a measure to the stage, until after the concrete head-gates had been placed at Andrade in 1906. Shortly after they were built, it was realized that the sill of the intake, which was 5 ft. below the bed of the river at the time, was too high, and this is what made it necessary to build temporary dams at low stages below the intake, in order to increase the head so as to divert enough water to supply the demands of Imperial Valley.

In July, 1910, the supply dropped to 746 sec-ft., only 395 sec-ft. reaching Imperial Valley. Much of this trouble was due to a silting up of the intake above the head-gates.

Both the United States and Mexico Interested.—A satisfactory and comprehensive solution of conserving, controlling, and using the waters of the Colorado River can only be reached by a joint commission of engineers from both sides of the International boundary line. It has already been too long deferred, and it is hoped that Mexican affairs will before long reach such a peaceful and prosperous state as to permit the consideration of the agricultural and other developments of the delta of the Colorado.

J. C. Allison,* Assoc. M. Am. Soc. C. E. (by letter).—A statement of developments, in the control work of the Colorado River protecting the Imperial Valley, since the paper was written in October, 1915, and since the discussion was published in September, 1916, is of value.

During January, 1916, there occurred the greatest flood ever recorded on the Lower Colorado. At Yuma the discharge was 210 000 sec-ft., or 60 000 sec-ft. in excess of any previous measurement. The restricted channel opposite the Imperial Valley intake carried the flood, without side-scouring and with no meandering of the stream. The channel in the vicinity of "Houses 5 and 6" continued to widen and meander to the extent of undermining the primary line of defense. Bank erosion through the restricted area was not increased in proportion to the stream-bed erosion by the increase in velocity. The writer's statement that "the less room the river is allowed for meandering, the greater its velocity and scouring action, and the easier it is to keep it under control", to which Mr. Ockerson has taken exception, is not only qualified here, but was again proved during the summer flood of 1916. The theory that the greater energy due to increasing velocity is expended on the bed of a stream, and not on the bank, is correct, applied, of course, to locations where the stream alignment is good and where local deflections do not occur. Local deflections will not occur where the stream bed is narrow and deep, but they do occur where it is wide and shallow and the formation of shifting sand-bars is made easily possible.

At the breach in the levees between "Houses 5 and 6", which took place in January, 1916, the Inter-California Railroad, the second line of defense, held the flood-waters, and kept them from entering the Alamo Channel near the site of the original 1906 break. The breach occurred again from lack of maintenance, and the incident does not alter the advisability of retaining this line of defense and the present method of revetment. Where the breach occurred, the levees were designed with land-side borrow-pits, and though the flood-water through the breach entered these land-side borrow-pits, it was easily controlled.

On the Lower Colorado River protective work, past experience demonstrates beyond a doubt that the land-side borrow-pits are more advisable. The writer has even gone so far as to propose in one instance the construction of a drain canal in the borrow-pit parallel with the protective levee, for the purpose of preventing the saturation of the lands adjacent to the levees. In no instance has a levee failure been traced to the presence of a continuous pit on the land side, so long as the width of the levee and the width of the berm are sufficient.

The irrigation system and the system of protective work are now

* Calexico, Cal.

Mr. Allison

in the hands of the Imperial Irrigation District. Improvements to the flood-protection system of the Valley have been begun, the greater part of which are in accordance with the recommendations set forth in the paper. An additional control-gate is now being constructed above the present Hanlon gate, as suggested by Mr. Sonderegger. The abandonment of the southerly end of the Hanlon Head-gate and the construction of additional openings in the rock to the north, are being planned to remedy the defects in that structure, as mentioned in the discussion.

The second line of levee-protection defense has been improved by placing a railroad track thereon throughout the whole extent of the Volcano Lake Levee, in accordance with early suggestions, and it is planned to open a quarry, in addition to the Hanlon Heading quarry, at the Volcano Lake end of the system. It is likewise planned to supply the district with sufficient equipment to facilitate present methods in maintaining the levee, namely, with rock revetment. In this regard, the suggestion made by Mr. Ockerson, which was carried out most efficiently and economically in his reconstruction of the Volcano Lake Levee, namely, that the rock facing should be laid by hand with some care, instead of being dumped at random from cars, is well founded in localities where the levee is not subjected to undermining from the river. This method of revetment is satisfactory for the entire second line of defense for Imperial Valley, but it is not practical on the primary line of defense and its extension, as proposed in the paper, on account of the difficulty of placing such rock to scour-line elevation.

The following is a résumé of the arguments and discussions concerning the theory and practices incident to the construction of a hydraulic-fill dam, as a means of diverting a stream flowing in an alluvial plain, which forms the principal subject of the paper.

The writer has made the statements that: "a direct relation exists between the character of the materials forming the bed and the velocities of the water", that "in refilling the bed, the materials are graded as to weight and deposited in direct relation to the velocities", and that in refilling the bed, the materials decrease "in weight as they approach the elevation corresponding with the lesser velocities." Mr. Ockerson cannot agree with this theory, and cites, as an example, the Mississippi River, in which bars composed of gravel larger than shingle exist well above the lower water, and deep concave bends opposite the bars contain no gravel whatever. Mr. Ockerson's citation is of a stream in which the channel is permitted to deflect from one side to the other, and in which increased areas are developed by side scour rather than by bed scour.

Such a stream as that cited by Mr. Ockerson is that section of the Colorado River at the site of the proposed dam to deflect the Colorado

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River water from its present course into the Bee River, back into the old channel of the Colorado. It is on account of such a condition that the writer has proposed the construction of a hydraulic-fill dam up stream some 12 miles, where the channel has been virtually confined to one bed for a great number of years, rather than where it has been shifting in different locations each year. The Mississippi River condition and that which makes possible the construction of the hydraulic-fill dam in the Colorado River at Hanlon Heading and Yuma, are in no wise similar. At Yuma, scour is not influenced by bed-rock, as stated by Mr. Ockerson, because bed-rock at this point is at a considerably lower elevation than the bed-scour line, and a sample of the materials deposited in the bed will reveal that they are graded as to weight, increasing as the scour line is approached.

The construction of a temporary dam in the Colorado River at Hanlon Heading became necessary once more in the summer of 1916. The Irrigation District constructed a rock-fill dam rather than a hydraulic-fill dam, similar to that built the year before, on account of the lack of adequate equipment to permit of the latter type, and because of the greater river discharge at the time the dam became necessary.

The river discharges, throughout the period when the dam was required, exceeded 30 000 sec-ft. The writer is of the opinion that with the proper equipment, as outlined in the discussion of the paper, the hydraulic-fill dam might have been placed successfully, at least with sufficient success to have restricted the channel area and to have developed the necessary head for diversion of sufficient water into the Imperial Valley Canal. As it was, a rock-fill dam was again placed across the Colorado River and later removed, at a total cost of more than $100 000.

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Paper No. 1388

TUNNEL WORK ON SECTIONS 8, 9, 10, AND 11, BROADWAY-LEXINGTON AVENUE SUBWAY, NEW YORK CITY*

By Israel V. Werbin, Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. Maurice Griest, John H. Madden, Robert H. Jacobs, T. Kennard Thomson, John H. Myers, H. G. Moulton, Robert Ridgway, C. V. V. Powers, Francis Donaldson, and Israel V. Werbin.

Synopsis.

This paper describes the work of excavating for and lining the tunnels for the new Lexington Avenue Subway between 53d and 106th Streets, in the Borough of Manhattan, City of New York. Inasmuch as this work involved the driving of tunnels with large and unusual cross-sections, through unsound rock, under busy city streets, and the contract placed on the contractor absolute liability, it has been thought worth while to record the methods used.

The tunnels were built to accommodate two and four tracks, the two-track structure having a cross-section of 32 by 16 ft., the four-track, double-deck structure, 32 by 32 ft., and the four-track structure with all tracks on one level, 60 by 16 ft.

The methods followed in excavating these tunnels through sound and unsound rock and through soft ground are described. In some cases, water was encountered in such quantities as to necessitate the use of compressed-air methods, in order to prevent the loss of materials and the settlement of the local track structure which had

* Presented at the meeting of September 20th, 1916.

previously been completed. The excavation for the four-track structure with the four tracks on the same level had to be made one track at a time, and the manner of making this excavation and protecting the finished part of the structure is described.

Information is also given as to drilling, loading and firing, timbering, plant, cost, etc.

In connection with the lining of the tunnel with concrete, the various types of forms are described; also the manner of handling and placing the concrete.

General.

The Lexington Avenue Subway is being built by the City of New York, under the supervision of the Public Service Commission for the First District of the State of New York, and, when completed, will be operated by the Interborough Rapid Transit Company in accordance with contracts entered into with it by the City of New York acting by the Public Service Commission.

For purposes of organization and construction, the route is divided into sections averaging in length about thirteen or fourteen city blocks.

Section 8 extends from 53d to 67th Street; Section 9 from 67th to 79th Street; Section 10 from 79th to 93d Street; and Section 11 from 93d to 106th Street. The total length of the four sections is 13 992 ft.

The approximate quantity of tunnel excavation made on the four sections was 250 000 cu. yd., and, at the contract prices, the cost of doing this part of the work was approximately $2 250 000. The total cost of all the work on the four sections was approximately $10 500 000, exclusive of track, station finish, and equipment.

The rock encountered in the course of the work on these sections was mostly Manhattan schist of varying degrees of hardness. The greater part of the tunnel being but a comparatively short distance below the street surface, the rock was often blocky, and occasional mud seams were encountered, requiring considerable timbering on Sections 8 and 9. The rock on Sections 10 and 11 was fairly good, and very little timbering was used for this part of the work.

General Description of Subway Structure.

Several different types of subway structure were used on the four sections. On Sections 8 and 9, the local and the express tracks are in separate structures. The local tracks are immediately below the street surface and the work was done by cut-and-cover methods; the express tracks are in separate tunnels below the local tracks. The distance from the street surface to the sub-grade of the express tracks varies from 60 to 100 ft., and the distance between the two levels from 10 to 55 ft., as shown on Plate IV.

The largest part of the structure on Sections 10 and 11 is double-decked, the express being placed immediately below the local tracks. This type prevails from 79th to 99th Street. In the station at 96th Street, the upper level is widened to permit the construction of the station platform (Plate IV). Going north from 99th Street, the local tracks in the upper level diverge until at 100th Street they have been offset sufficiently to clear the express tracks in the lower level. The four tracks then continue northward on approaching grades until at 103d Street they are on the same level, as shown on Plate IV.

Excavation for Two-Track Tunnel, Sections 8 and 9.

Excavation in Sound Rock.—The excavation for these tunnels through sound rock did not present any exceptional features. The section was taken out with a center top heading and one or two benches, depending on the height of the heading. The center heading was about 9 by 14 ft., the drilling requiring from 24 to 32 holes, depending on the hardness of the rock. Work was carried on continuously for 24 hours per day, with three drilling shifts. The forces were organized so that an advance of about 5 ft. would be made daily. The details of the drilling, loading, and firing are shown on Plate V.

Excavation in Unsound Rock.—In a considerable part of the work, the rock was hard but blocky, and would not permit of making the excavation for the full section without timbering. In most of these cases it was generally possible to excavate enough to permit the erection of the structure for one track without timbering, and after this part had been completed and the rock above had been caught up, the remainder of the section was excavated and the structure completed. (Fig. 1.)

Where the rock was seamy and disintegrated, however, the top side-drift method of tunneling was generally used, segmental timbering being put in, as shown on Plate VI. The side-drifts were generally about 7 by 11 ft. The wall-plates and timbering were set so as to be entirely outside the neat line of the subway structure and there would be no timbers bedded in the concrete lining inside the neat lines. Where soft ground was encountered, poling boards were driven ahead over the sets. The sets were placed about 2 ft. from center to center, 12 by 12-in. yellow pine timbers being used.

In the work on Section 8, where segmental timbering was used, the center pieces of the sets were posted up from the bench, and, as the latter was removed, new posts down to sub-grade were put in, as shown by Fig. 3. In removing the bench, the holes were drilled close together, and light charges of explosives were used in order to minimize the possibility of knocking out the posts. Toward the close of the job, in order to safeguard further this feature of the work, a long, continuous girder was blocked up under the center cap, and this was kept in place until the bench had been removed and new posts, down to sub-grade, had been placed. (Fig. 2.)

On Section 9, after the top heading had been removed, the contractor placed two (and in some cases three) pairs of 24-in. I-beams longitudinally on the bench, and the caps were posted up from these beams. (Fig. 4.) The beams were about 30 ft. long, and were spliced so as to develop their full strength. As the bench was removed, posts were placed under the beams down to the sub-grade. This system of timbering had the advantage that the posts under the caps did not have to be moved in order to make the excavation for the bench. (Fig. 5.)

Often, where the ground was heavy enough to require timbering, water was encountered, and it was desirable to keep this water away from the concrete lining until it had had time to set. Accordingly, the space between and back of the sets was packed with concrete, and later grouted under about 90 lb. pressure. This procedure was effective in keeping out most of the water, and also served to stiffen up the timbering. The final lining was placed tight up against the timber and concrete previously placed, and again grouted if necessary. (Fig. 6.)

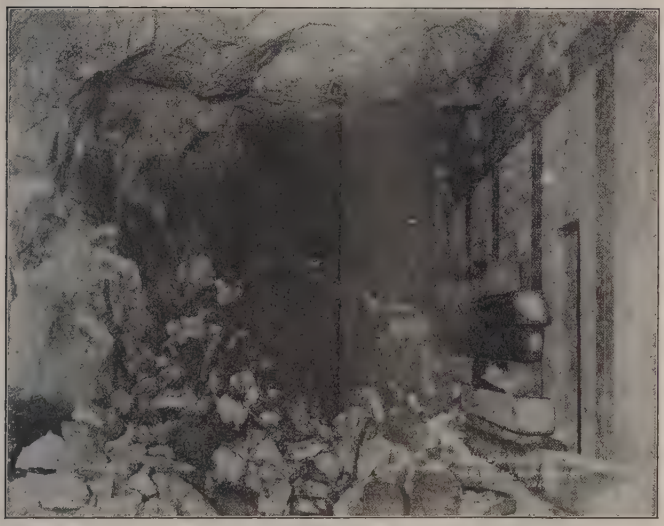


FIG. 1.—CONSTRUCTION IN POOR ROCK TUNNEL. HALF OF SECTION BUILT AT A TIME, WITHOUT TIMBERING.

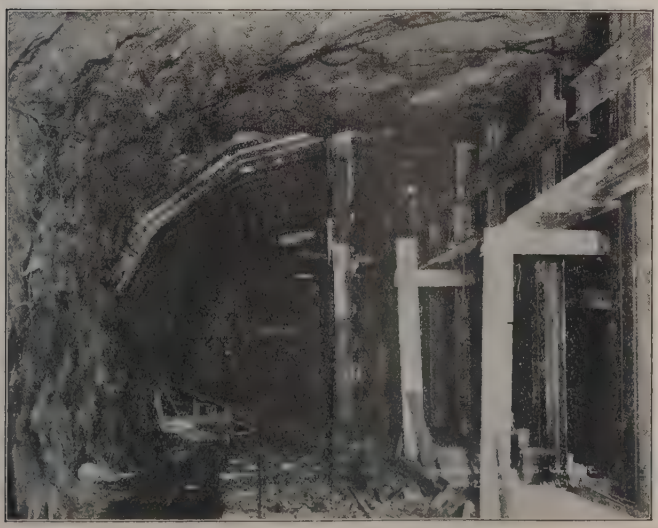


FIG. 2.—SEGMENTAL TIMBERING USED ON SECTION 8; ALSO TYPES OF CENTER LINE STEEL USED IN THE TUNNEL.

Excavation in Compressed Air.—At 57th and 75th Streets, streams originally crossed Lexington Avenue, and when the excavation was being made for the express tunnels at these places, considerable water was encountered. The contractors had completed the structure for

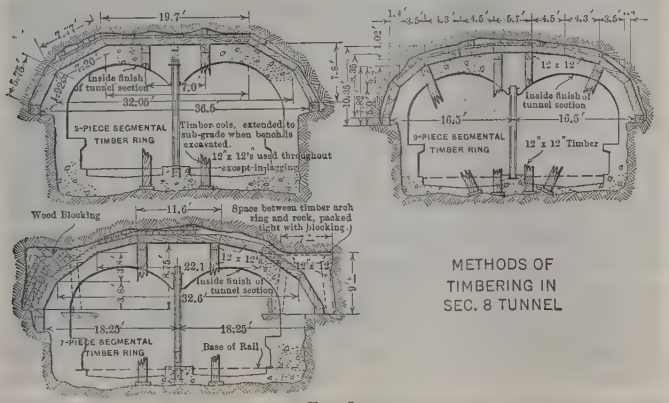


FIG. 3.

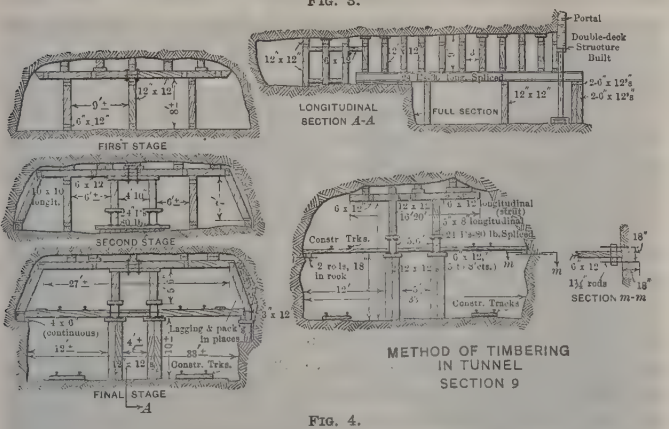


FIG. 4.

the local tracks in the upper level at these points before the work in the express tunnels was undertaken. This had been done in spite of the fact, pointed out to them, that settlement in the local track structure might result if soft ground carrying water was encountered

in the excavation for the express tunnel. As the excavation approached the sites of the old streams at 75th Street, the top of the section ran out of rock, and at 57th Street there were only a few feet of rock cover over the heading. In both places considerable water ran into the excavation, and the water, carrying with it some of the material between the two levels, caused considerable settlement in the local track structure, as well as some settlement in the buildings adjacent to the work. An attempt was made to control the settlement by blocking up the roof of the local track structure and by supporting the buildings temporarily on needles, also by pumping the water through driven wells from the local track floor, but neither of these methods proved satisfactory. It was decided finally to proceed with the work under air pressure, by which method the excavation was completed successfully without any appreciable loss of materials.

The details of the locks and bulkheads used at 57th Street are shown on Fig. 7. The air pressure varied from 8 to 16 lb. Concrete bulkheads, 3 ft. thick and reinforced with structural steel or iron rails, were built into the finished section of the tunnel. A groove about 6 in. deep was cut into the concrete lining of the tunnel, and the bulkheads were bonded into it. The lock and the pipes for air, drainage, etc., were built into the bulkheads.

The excavation under air pressure was confined to the removal of the top heading. After the timber sets had been placed and the space between and back of the timbers had been concreted and grouted, the bench was removed in free air.

At 57th Street, where the section did not run out of rock, the excavation under air pressure was made by the top side-drift method, as described for the unsound rock tunnels on Section 8.

At 75th Street, the top of the heading was driven through soft ground, and poling boards were used. The general method of procedure (Fig. 8) was about as follows: The excavation was started at the center and carried over to the sides. Poling boards, about 4 ft. long, were driven over the caps, and, as the excavation was carried from the center to the sides, breast boards were placed against the face of the heading and under the ends of the poling boards, until the excavation had been widened enough to permit the placing of another cap, after which the breast boards were removed and the

FIG. 5.—METHOD OF TUNNEL TIMBERING USED ON SECTION 9.

FIG. 6.—SPACE BETWEEN TIMBER SETS CONCRETED.

operation was repeated. After three or four caps had been placed in this way, the excavation was widened out for the wall-plates and side-legs. Where considered necessary, lagging was placed and braced, as shown on Fig. 8.

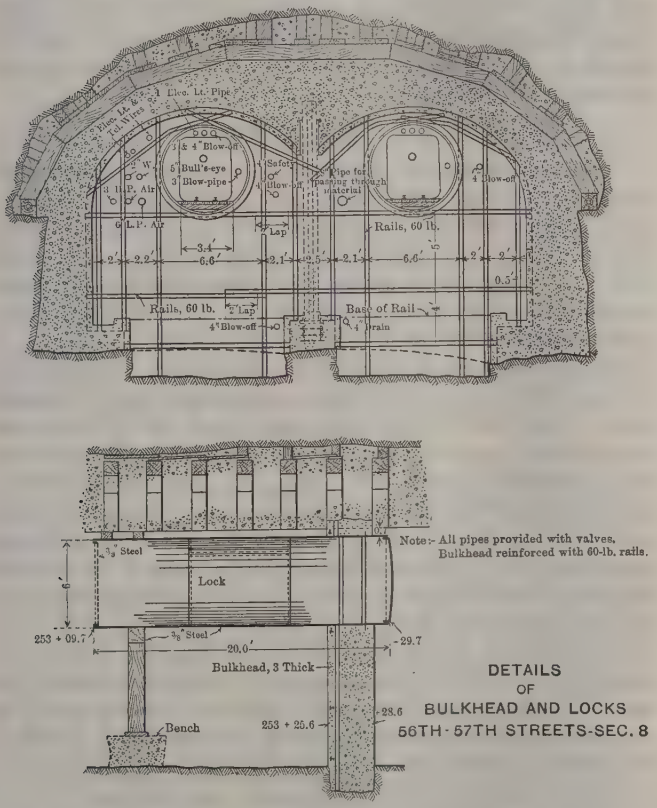


FIG. 7.

In some cases, the soft ground showed a tendency to flow and work its way in between the roof and breast boards, and in such cases the cracks were stuffed with hay. At certain times there was considerable difficulty in holding the air, and for this reason it often

became necessary to caulk the cracks between the timbers with oakum and clay.

Working Shafts.—There were working shafts, for handling the excavated material from the express tunnels on Sections 8 and 9, at 56th, 62d, 68th, 74th, and 78th Streets, the distances between shafts being 1 605, 1 662, 1 518, and 1 057 ft., respectively. At 62d and 74th Streets, the hoisting was done with electric traveling cranes; derricks operated by electric hoists were used at the other shafts.

Roadway Support at Portals, Sections 10 and 11.

The tunnels on Sections 10 and 11 were not continuous, there being stretches of open-cut work between them. The excavation was generally started at the cut-and-cover part of the work, and then extended into the tunnels. The depth of the cuts between the portals varied from 50 to 70 ft., and it became evident that some method of supporting the roadway, different from the usual one of posting up from below, would have to be used.

Two general methods of supporting the roadway were used. By the first, 48-in. girders, 60 ft. long, were placed about 40 ft. apart across the full width of the trench, with the ends resting on the rock or on cribbing built up from the rock. Then 26-in. I-beams were run longitudinally over the cross-girders, and the roadway was blocked up from them. After the excavation was completed, the cross-girders were posted up from below, as shown by Fig. 9. As an additional precaution, 48-in. girders were placed longitudinally in the roadway alongside of the curb, and the cross-girders were connected with them by suspender rods and plates. The surface girders were spliced with a joint designed to develop the full strength of the girder before it would fail. One end of the longitudinal girder was supported by being blocked up from a cross-girder that was already posted up from below, and the other end rested on the undisturbed portion of the roadway. As the excavation advanced, the longitudinal girders were moved ahead, or an additional girder was spliced on.

By the second method, the longitudinal girders just described were used, but 26-in. I-beams running crosswise were suspended from them, and the roadway was blocked up from these cross-beams. The beams were placed about 10 ft. from center to center, and, after the excavation below was completed, the beams were posted up and the longitudinal girders moved ahead. (Fig. 10.)

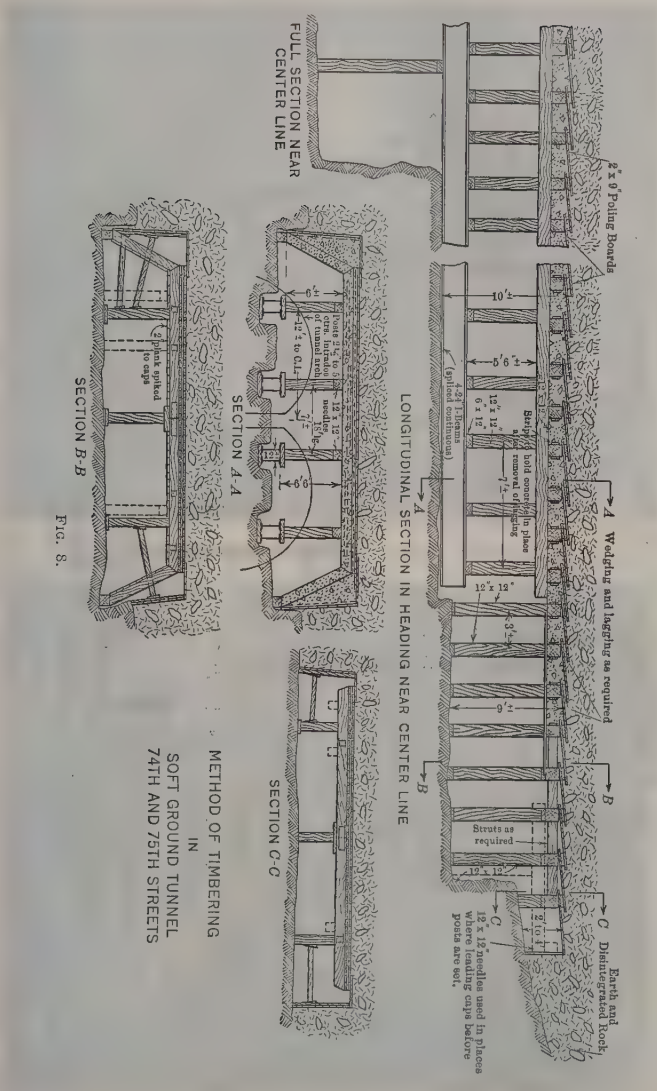

FULL SECTION NEAR CENTER LINE
2" x 9" Poling Boards
10'±
5'6"±
Strips to hold concrete in place after removal of lagging
12" x 12"
6" x 12"
7'±
4-2½" I-Beams (spliced continuous)
A
Wedging and lagging as required
12" x 12"
3'±
9'±
B
Struts as required
12" x 12"
C
Earth and Disintegrated Rock
12" x 12" needles used in places where leading caps before posts are set.
LONGITUDINAL SECTION IN HEADING NEAR CENTER LINE
6'±
Posts 2½ to 5 ctrs. intrados of tunnel arch
12" x 12" needles 15 lg.
12'± to C.L.
12"
6'6"
SECTION A-A
2 plank spiked to caps
SECTION B-B
SECTION C-C
METHOD OF TIMBERING IN SOFT GROUND TUNNEL 74TH AND 75TH STREETS

FIG. 8.

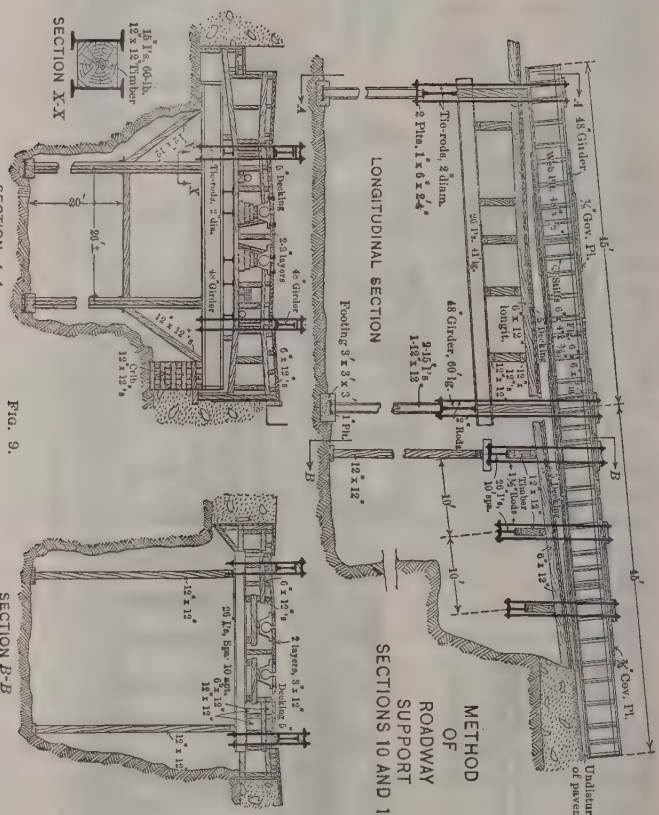

METHOD
OF
ROADWAY
SUPPORT
SECTIONS 10 AND 11.
LONGITUDINAL SECTION
SECTION X-X
SECTION A-A
SECTION B-B
Undisturbed portion of pavement.
48" Girder
Tie-rods, 2" diam.
2 Plts. 1" x 6" x 2'-4"
48" Girder, 60' lg.
2-15 I's
1-12" x 12"
Footing 3' x 3' x 3'
1" Pl.
12" x 12"
Timber
Rods
Cov. Pl.
15" I's, 60-lb.
12" x 12" Timber
5" Decking
2-3" layers
48" Girder
Crib
2 layers, 3" x 12"
Decking 5"
26 I's, Spa. 10 ant.

FIG. 9.

FIG. 10.—ROADWAY SUPPORTED ON CROSS-BEAMS, SECTIONS 10 AND 11.

FIG. 11.—EXCAVATION IN DOUBLE-DECK TUNNELS.

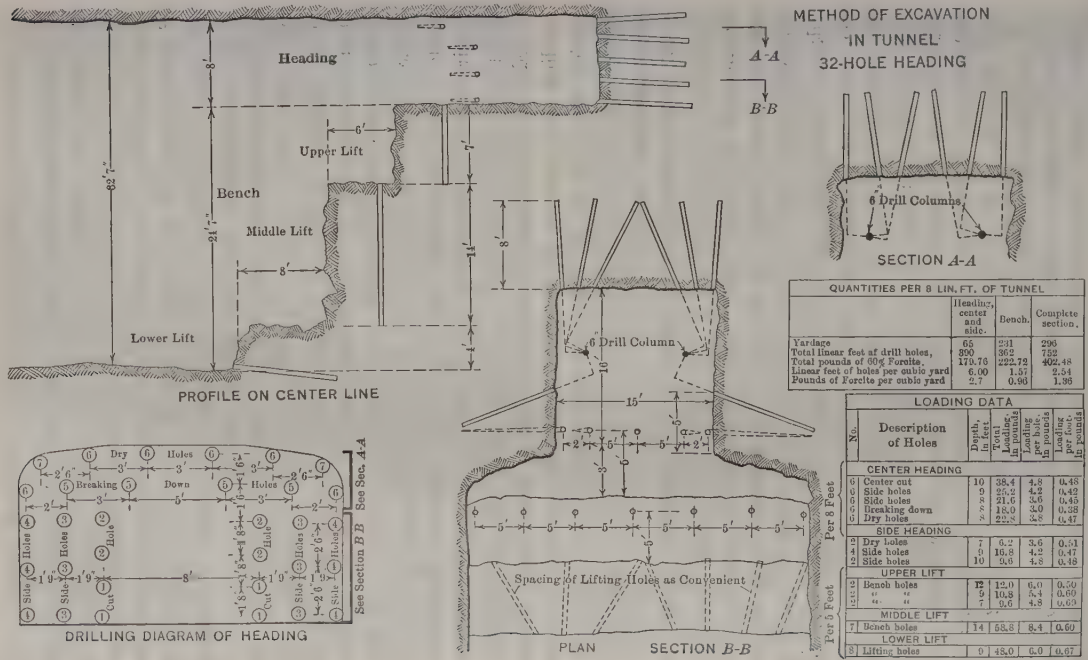
EXCAVATION FOR DOUBLE-DECK TUNNEL, SECTIONS 10 AND 11.

The double-deck tunnel is 31 ft. 8 in. wide and 32 ft. 7 in. high. The section was generally taken out with a center top heading and three benches. An advance of from 5 to 8 ft. of heading was made at each fire. About 8 lin. ft. of drill holes were required per cubic yard of heading, and 1.6 ft. per cubic yard of bench. The approximate arrangement of drill holes, and the loading and order of firing are shown on the heading diagrams, Figs. 12 and 13.

The excavation was started with a 32-hole heading (Fig. 12), but, owing to the severe concussion following the blasts, it was found advisable to increase the number of holes to 48 (Fig. 13) until the excavation was sufficiently advanced beyond the portal, so that the effect of the blasting would not be so severe. The depth of the cut was reduced from 8 to 6 ft., and the spacing of the holes in the bench was reduced from 5 to 3 ft. This new arrangement required much more drilling (4.7 lin. ft. per cu. yd. of excavation as compared with 2.5 lin. ft. previously used), but was efficacious in reducing the concussion and the subsequent vibration of adjacent buildings and breaking of windows. After the excavation had been carried about 250 ft. beyond the portal, there was little concussion noticed on the street, and the drilling was brought back to the 32-hole heading. (Fig. 11.)

In the prosecution of this part of the work, the aim was to make an advance of from 5 to 8 ft. in 2 days. In general, the center heading and the middle bench were fired on one day, and the upper and the bottom benches on the next. The advance in the heading varied from 5 to 8 ft., depending on the character of the rock; the sides and the benches were taken out about 5 ft. at a time, an extra shot being occasionally taken out of the sides and benches, so that the progress would keep up with that of the heading. The work was done with one mucking and two drilling shifts working as follows: Mucking gang, 10 A. M. to 7 P. M.; drilling gangs, 6 A. M. to 2.30 P. M., and 4.30 P. M. to 12.30 A. M. The blasting was done before the mucking gang came on (between 7.00 and 10.00 A. M.), and from 12 M. to 1 P. M., when the mucking gang stopped for lunch.

The best monthly progress in excavation in this tunnel was 100 ft. of full section, or about 3 700 cu. yd. of excavation within neat lines.



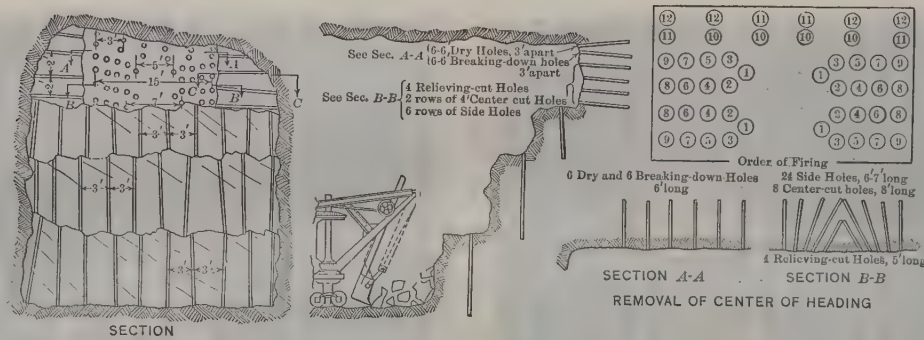
QUANTITIES PER 8 LIN. FT. OF TUNNEL

	Heading, center and side.	Bench.	Complete section.
Yardage	65	231	296
Total linear feet af drill holes,	390	362	752
Total pounds of 60% Forcite,	170.75	222.72	402.48
Linear feet of holes per cubic yard	6.00	1.57	2.54
Pounds of Forcite per cubic yard	2.7	0.96	1.36

LOADING DATA

No.	Description of Holes	Depth, in feet.	Total Loading in Holes.	Loading per hole, in pounds	Loading per foot, in pounds
	CENTER HEADING				
6	Center cut	10	38.4	4.8	0.48
6	Side holes	9	37.2	4.2	0.42
6	Side holes	8	21.6	3.6	0.45
5	Breaking down	7	18.0	3.0	0.35
6	Dry holes	8	22.8	3.8	0.47
	SIDE HEADING				
2	Dry holes	7	6.2	3.6	0.51
4	Side holes	9	16.8	4.2	0.47
2	Side holes	10	9.6	4.8	0.48
	UPPER LIFT				
2	Bench holes	12	12.0	6.0	0.50
8	" "	9	10.8	5.4	0.60
2	" "	7	9.6	4.8	0.69
	MIDDLE LIFT				
7	Bench holes	14	58.8	8.4	0.60
	LOWER LIFT				
3	Lifting holes	9	48.0	6.0	0.67

FIG. 12.


SECTION
See Sec. A-A 6-6 Dry Holes, 3' apart
6-6 Breaking-down holes 3' apart
See Sec. B-B 4 Relieving-cut Holes
2 rows of 4' Center cut Holes
6 rows of Side Holes
Order of Firing
6 Dry and 6 Breaking-down Holes 6' long
24 Side Holes, 6'-7' long
8 Center-cut holes, 5' long
4 Relieving-cut Holes, 5' long
SECTION A-A
SECTION B-B
REMOVAL OF CENTER OF HEADING

LOADING DATA.

No.	Holes Description	Depth, in feet.	Total loading, in pounds.	Loading per hole in pounds.	Loading per foot, in pounds.
	CENTER HEADING.				
4	Relieving holes	5	7.0	1.75	0.35
8	Center cut	8	25.6	3.20	0.40
24	Side holes	6.5	46.8	1.95	0.30
6	Dry holes	6	10.8	1.80	0.30
6	Breaking-down	6	10.8	1.80	0.30
	SIDE HEADING.				
20	Side holes	"	56.0	2.80	0.35
	BENCH.				
10	Bench holes	6	21.0	2.10	0.35
10	" "	12	42.0	4.20	0.35
10	" "	10	35.0	3.50	0.35

QUANTITIES PER 8 LIN. FT. OF TUNNEL.

	Heading, centre and sides.	Bench	Complete section.
Yardage	67	105	222
Total linear feet of drill holes.	472	560	1032
Total pounds of 60 % Forcite.	157	306	463
Linear feet of holes per cubic yard.	8.3	3.4	4.7
Pounds of Forcite per cubic yard.	2.8	1.2	1.6

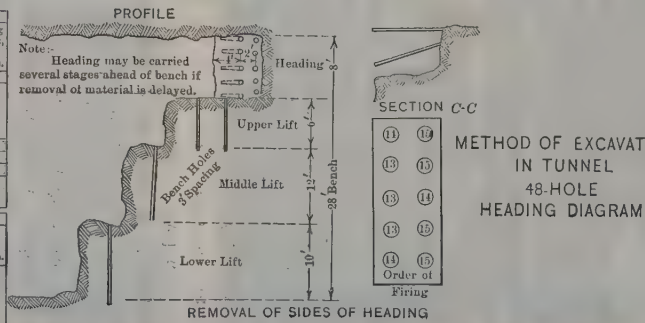


FIG. 13.

Trimming at Portal and Widening Out for Station Platforms.

Generally, only a few feet of rock cover was found at the portals, and, as this rock was seamy and disintegrated, the excavation was started about 5 ft. below the line of the finished roof and gradually brought up to grade in about 100 ft. At the 95th Street portal, where the work was in the station, no attempt was made at first to excavate for the station platforms, the contractor intending to come back later, after the lower-level structure had been completed, and do this part of the work.

Before the portals were trimmed and the excavation was widened out for the station platforms, the plans were changed so as to substitute steel columns with longitudinal beams or girders on top for the reinforced concrete center wall originally called for. This change of plan greatly simplified the work, and lessened the risk of excavating for the station platforms and trimming at the portals.

As has been noted previously, the rock at the portals was very poor (Fig. 14), and particular care had to be taken not to expose too much of it at a time. The structure for the lower level was first completed (Fig. 16), and also for the upper level inside the portal, where the excavation had already been made to the full section. The work at the center was trimmed carefully to the neat line for about 20 ft., commencing at the point where the upper level had already been completed, and working toward the portal (Fig. 15). While the center was being trimmed, the rock on the sides was made secure by posting or cribbing from the roof of the structure of the lower level. The center line steel and concrete wall was then built for about 15 ft., and carried up solid to the rock above. After the rock was thus caught up at the center, the roof on one side was trimmed and the arch placed (Fig. 17), after which the other side was completed. In this way, by working from the finished structure toward the portal, trimming small stretches at a time, and keeping the exposed rock in the roof well supported, the trimming at the portals and the widening out for the station platforms were accomplished successfully.

Excavation, 99th to 102d Street.

Between 99th and 102d Streets, the transition is made from a double-deck tunnel structure to one with four tracks on the same

FIG. 14.—TYPICAL CONDITION OF ROCK AT THE 95TH AND 96TH STREET PORTALS.

FIG. 15.—CENTER WALL CONSTRUCTION, 96TH STREET STATION.

level. The span of the tunnel at 99th Street is 32 ft. (Fig. 19), and this gradually increases until, at 100th Street, the span is 60 ft. (Plate IV). At this point, the local tracks have been offset sufficiently to clear the two center tracks in the lower level, and continue northward in two separate one-track tunnels, the rock core between them (over the center tracks) not being removed. (Plate IV.)

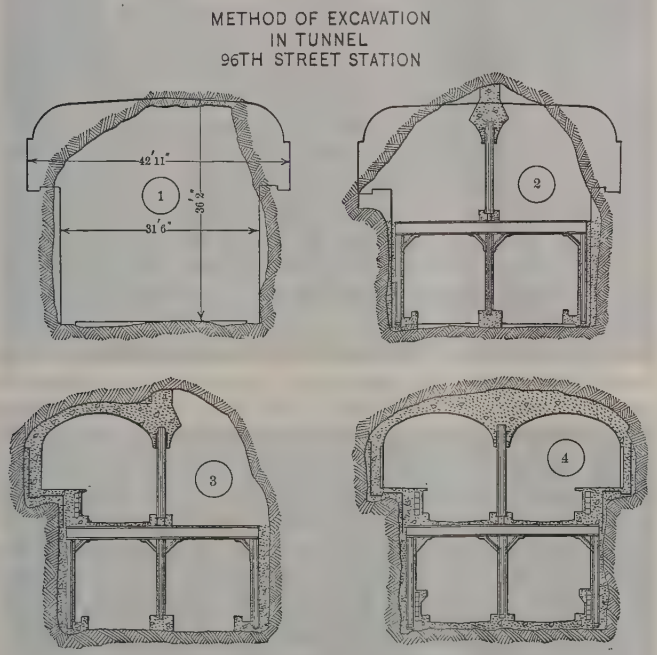


FIG. 16.

In making the excavation for the widening north of 99th Street, a section approximating that of the typical double-deck tunnel was first taken out, and the structure for the lower level completed. The roof was then trimmed at the center to permit the erection of the center steel, and, after the rock was solidly blocked up from the tops of the columns, the excavation was increased to its full width without requiring any additional timbering, except for occasional posting under unsound rock.

The most difficult part of the tunnel work on Lexington Avenue was the excavation for the transition between 100th and 102d Streets. The tunnel for this stretch had a span of 60 ft., and the rock was cut up and disintegrated, and would hardly stand without support for the span of one track (15 ft.). The situation was complicated further by the fact that the plans did not contemplate the excavation of the rock core over the center tracks (lower level) and between the local tracks on the sides.

The general scheme of prosecuting the work was to complete first the excavation for the center tracks, build this part of the structure, and, after the rock over the roof was properly caught up, to make the excavation for the local tracks on the sides. The excavation for the two center tracks was continued northward to about 100th Street without encountering any special difficulties. Going north from 100th Street, the rock began to get bad, and it became evident that it would not be safe to make the excavation for the full width of the two-track structure without the use of timbering. As the contractor desired to continue the use of his air shovel for mucking, and as the use of the ordinary methods of timbering requiring posting would interfere with the operation of the shovel, it was decided to narrow the excavation to a width of about 18 ft., this being sufficient to permit the use of the air shovel. The excavation was confined to one side, so that, when completed, the structure for one track could be built. It was hoped that the rock would arch for this small span, and that by this procedure there would be no necessity for timbering. This scheme was successful for the greater part of the work, but occasionally there were places where, even with such a small span, it was necessary to support the roof. In these cases, in order to permit the use of the air shovel for mucking, the timbering had to be placed so that it would not interfere with the shovel or with the construction that was to follow later.

The general manner of putting in roof supports in these cases (Fig. 21) was about as follows: When unsound rock was encountered, the excavation was advanced with a small heading (8 by 8 ft.), the rock in the heading being secured temporarily by posting. At a point where the excavation to the neat line had been completed previously, recesses were cut in the side rock above the line of the roof of the finished structure, and 24 or 26-in. I-beams were placed

Fig. 17.—Half of Roof in 96th Street Station Concreted.

Fig. 18.—Rock Conditions at 102d Street Portal.

FIG. 19.—DOUBLE-DECK OPEN-CUT STRUCTURE AT 98TH STREET.

FIG. 20.—CONCRETE BUTTRESSES AGAINST 102D STREET PORTAL.

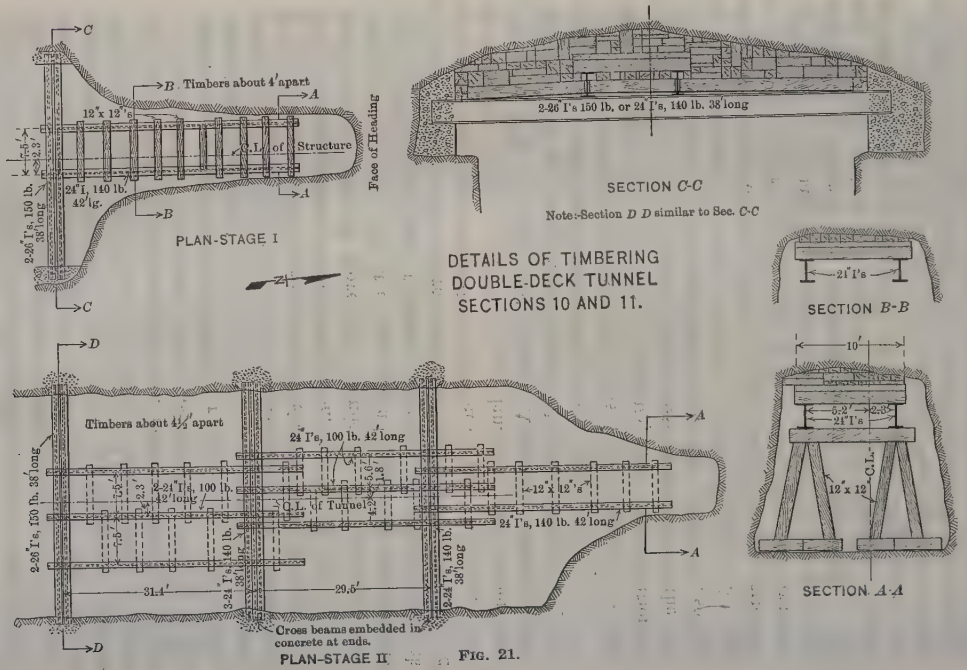

C
B
Timbers about 4' apart
A
12"x 12"s
C.L. of Structure
Face of Heading
24 I's, 140 lb. 42' lg.
2-26 I's, 150 lb. 38' long
PLAN-STAGE I
2-26 I's 150 lb. or 24 I's, 140 lb. 38' long
SECTION C-C
Note:-Section D D similar to Sec. C-C
DETAILS OF TIMBERING
DOUBLE-DECK TUNNEL
SECTIONS 10 AND 11.
21 I's
SECTION B-B
D
Timbers about 4½' apart
24 I's, 100 lb. 42' long
2-26 I's, 150 lb. 38' long
2-24 I's, 100 lb. 42' long
C.L. of Tunnel
12"x 12"s
24 I's, 140 lb. 42' long
2-24 I's, 140 lb. 38' long
31.4'
29.5'
Cross beams embedded in concrete at ends.
PLAN-STAGE II
10'
5.2'
2.35'
24 I's
C.L.
12"x 12"
SECTION A-A

FIG. 21.

crosswise, with the ends of the beams concreted into these recesses. A temporary horsehead was then set up in the heading, and longitudinal beams were placed, supported on the horsehead and the cross-beams. The rock was then blocked up from the longitudinal beams, and the section was increased to its full width, after which another set of cross-beams was put in just below the horsehead, and the longitudinal beams were brought to bear on this new set. The small heading was then advanced another 50 ft., and the operation repeated.

These systems of timbering were used for about 200 ft. of the tunnel, and though they were efficient for the purpose of supporting the rock and permitting the use of the shovel, they interfered seriously with the progress of the work, as it took about a week to put in about 50 ft. of the timbering, during which period the work of excavation had to be discontinued. For this reason, toward the close of the job, the use of the air shovel was abandoned, the excavation was confined to only one track at a time, and all loose rock was carefully removed. In some cases it became necessary to remove the rock as high as 12 ft. above the roof line, and though the contractor did not receive any payment for the excavation outside of the neat lines, he preferred to remove all loose and dangerous rock rather than attempt to hold it in place with timber. A powerful electric searchlight was kept on the job at all times, and examinations of the roof were made several times a week, all loose rock being removed.

The rock throughout this stretch was very poor, and, as very little timbering was used, it was necessary that the excavation be followed by the construction as soon as possible, for, though the roof was carefully scaled, the constant blasting loosened up the rock, so that it was inadvisable to leave it exposed and unsupported for any considerable length of time. Besides, the contractor was not permitted to widen the center excavation for the two tracks until the structure for the one track had been completed and the rock over it had been caught up, or to make the excavation for either local track on the side until the roof of the structure for the adjacent express track had been completed and the rock over it had been safely caught up.

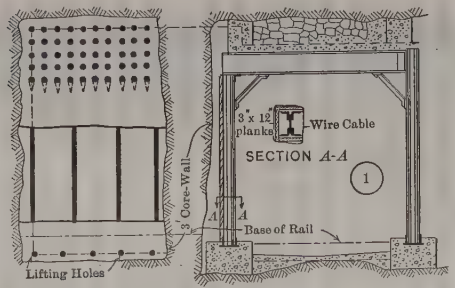
At first, the space over the roof was filled with hand-packed rock, but this proved unsatisfactory, for, when the excavation was made for the local tracks on the sides, the hand-packing was not sufficient to hold the rock core over the express tracks in place. Horizontal

seams started to develop in the core, and the voids in the hand-packing as well as the cracks in the rock core were then grouted with Portland cement grout to prevent bringing concentrated loads on the roof. After this, where the excess space was not too large, or where the rock was very poor, the roof concrete was carried solid to the rock above; where the rock broke very high, continuous longitudinal concrete walls, from 2 to 4 ft. thick, were built over each of the partition walls of the structure and carried solid to the rock above, and the space between the walls was filled with hand-packing (Plate VII).

No attempt was made to excavate for the local tracks on the sides until the adjacent express track structure had been completed and the overlying rock had been caught up. The plans called for an emergency exit to the surface at 100th Street, with a cross-drift over the express tracks connecting the two outside tracks. The contractor used this shaft and drift as working points from which to make the excavation for the local tracks.

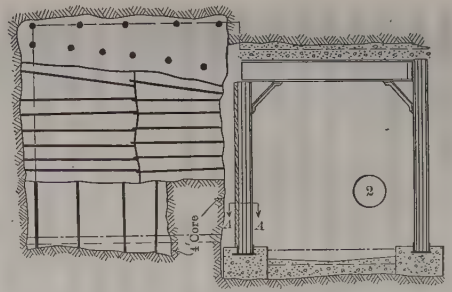
While the excavation for the local tracks was being made north and south from 100th Street, work was also started at the 102d Street portal. At this point, the structure consists of four tracks on the same level, the tunnel having a cross-section of 60 by 17 ft. The rock as exposed at the portal was faulted and broken up, and it was clear that it would not be safe to excavate for more than one track at a time. (Fig. 18.) The excavation was first made for the west, outside (local) track, and carried south about 75 ft. from the portal. A cross-drift, about 20 ft. wide, was then carried to the east neat line, and the excavation for the east, outside (local) track was extended north to the portal. After the structure for this part of the excavation had been completed and the roof had been concreted solid to the rock above, the excavation for the two center tracks was made, but prior to this, and as an additional precaution against a movement of the rock over the portal, concrete buttress walls (about 3 ft. thick) were built up against the face of the portal, and rested on the roof of the finished structure in the open cut beyond the portal. Three buttress walls were built, one over each of the partition walls of the subway (Fig. 20).

No special difficulties were encountered in making the excavation for the local tracks on the sides. Precautions, however, had to be taken to protect the adjacent finished structure (Fig. 22).

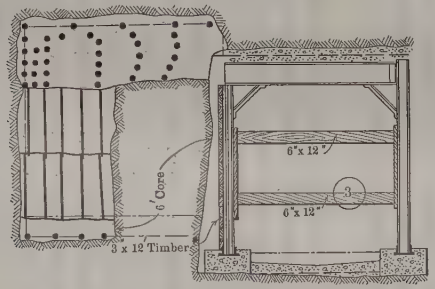


HAMMER-CUT SYSTEM

Core-wall from sub-grade to Roof.
Steel protected by vertical planks.



Note:- Center heading drilled laterally,
Top of section drilled longitudinally.



Steel protected by horizontal planks (longitudinal)

METHODS OF PROTECTING STEEL
IN
4-TRACK TUNNEL
100TH-102D STREETS

FIG. 22.

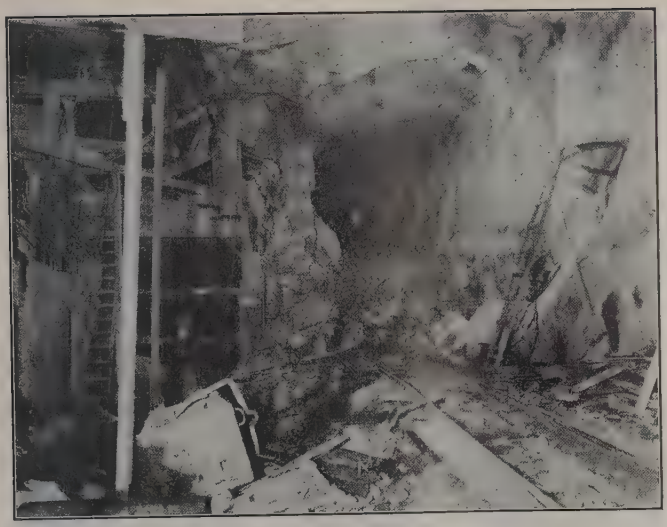


FIG. 23.—EXCAVATION FOR LOCAL TRACKS IN FOUR-TRACK TUNNEL.

FIG. 24.—SUBWAY STRUCTURE DAMAGED BY BLASTING.

FIG. 25.—DAMAGED COLUMNS REMOVED AND BEAMS SUPPORTED BY 12 BY 12-IN. POSTS.

FIG. 26.—BEAM PLACED BETWEEN ROOF BEAMS TO STIFFEN THEM AND DISTRIBUTE THE LOAD ON THE SIDE-WALL.

At first, the excavation for each local track was made as a separate tunnel, leaving a core-wall, from 3 to 4 ft. thick, between the excavation and the finished structure. The core-wall was kept about 25 ft. behind the heading, and was shattered so much by the adjacent blasting that it could be easily removed with little drilling and blasting. This method was soon abandoned, as considerable damage was done to the finished structure, by pieces of rock being blown against the columns during the blasting, and also by the rock of the core-wall being jammed against the columns. The columns were protected by timber planks, and, in some cases, by encasing them in the concrete partition walls called for by the plans. Though in some instances the concrete was injured by the blasting, the damage done to the structure was less serious than in the cases where concrete was not put in.

Very good results, as far as protecting the steel was concerned, were obtained by drilling the holes laterally and firing the rock away from the finished structure. Practically, this consisted of taking out a center heading and later removing the top of the section with breaking-down holes. When the bench was removed, a 4-ft. core-wall was left temporarily between the excavation and the finished structure. This method was soon abandoned, as its use generally required platforms or scaffolding on which to set up the drills.

The final and most successful method used was to place a continuous bulkhead of 3-in. planks between the steel and the rock to be excavated, and to brace the columns and partition wall over to the center wall with two 6 by 12-in. braces. The excavation was made with a small top heading and three benches, a 6-ft. core-wall being left temporarily between the bench and the columns. The core was really a part of the bench that had not been drilled, but it was shattered so much by the other blasting that it could be removed later with very little drilling and blasting (Fig. 23).

Considerable damage was done to the structure adjacent to which blasting was done (Fig. 24). The contractor was required to cut out and replace in a satisfactory manner about 200 ft. of 16-in. reinforced concrete wall and columns. Prior to removing the columns, the roof beams were posted up with 12 by 12-in. timbers to prevent any settlement (Fig. 25). In some cases the ends of the center track roof-beams were badly damaged as well as the columns, and it was not con-

sidered safe to attempt to remove them. In these cases a 12-in. I-beam was placed between the roof-beams to stiffen them and to distribute the load over the concrete wall instead of directly on the columns when the side-walls for the local track were placed (Fig. 26). In some cases, also, where jack-arches were used in the roof, they were badly damaged. As it was not considered safe to remove these arches and replace them, 6-in. cross-beams were introduced in the bottom of the arch and grouted, as shown in Fig. 27.

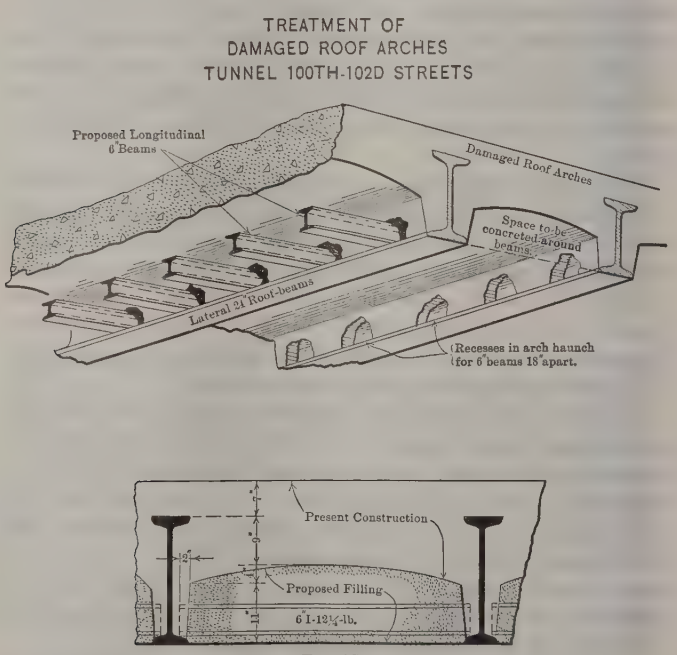


FIG. 27.

Some repair work was also necessary to the local track structures at 56th and 74th Streets, where they had settled on account of the excavation for the express track tunnel. North of 74th Street, considerable voids were found under the track floor. The ground at this point was a soft alluvial soil, and it did not seem likely that any satisfactory results could be obtained by grouting. Accordingly, the

contractor was ordered to re-build the footings under the columns and carry them down to the soil. The track floors were also re-built. South of 74th Street the void space was found under the floor for about 100 ft. of the structure, the voids in some cases being as great as 1 ft. The work in the express tunnel had drained the soil at this point so that it was compact and dry when repairs were to be made. Holes were placed in the local track floor about 10 ft. apart, and the voids were filled with grout. After this was done, investigating holes were again made through the floor and grout, and failed to disclose any void space. At 56th Street, when settlement began to take place, the contractor immediately grouted the space under the local track floor, and this did much toward checking the settlement. Later, when test holes were made through the local track floor, the grout, in some cases, was found to be 20 in. thick, and, as this rested on good soil, it was not necessary to make any material repairs, as it was possible to work up a new grade with the structure in its final location.

Handling of Muck.

In the work on Sections 8 and 9, the mucking was done by hand; on Sections 10 and 11, air shovels were used. The excavated material was loaded into large iron buckets, commonly known on the work as "battleships", and was not handled again until dumped into the scows for disposal. The flat-top trucks used were fitted with cradles to receive the bottom of the "battleships".

Plant.

The contractor's plant was operated by compressed air and electricity. The power-plant for the air was at 96th Street and East River. Electric current was furnished by the New York Edison Company.

Compressed-Air Plant.—The compressed-air plant consisted of five Ingersoll-Rand compressors, each with a capacity of 2 100 cu. ft. of free air per min., and five General Electric synchronous motors, each of 350 h.p., 6 600 volts, and 24.7 amperes. Five receivers were used, two 6 by 18 ft., and three 5 by 15 ft. The air was brought to Lexington Avenue through 96th Street in a 10-in. pipe line. The pipe line was extended north and south of 96th Street to 53d and 106th Streets, and connections were made at the shafts as required. The air was com-

pressed to a pressure of 100 lb., and was delivered in the headings at about 90 lb.

Shovel.—Four 38-ton Marion shovels, Model No. 40, with 1-yd. buckets, were used on Sections 10 and 11. The shovels were provided with 18-ft. booms when working in the double-deck tunnels, but these were cut down to 15 ft. when the shovel was used in the single-deck tunnel.

Hoists.—The electric hoists used at the surface at the shafts were generally Lambert, direct-current, 60-h.p. 230 volts, and 220 amperes. The hoisting engines used below for hauling the mucking cars were Lambert 7 by 10-in. double-drum, and were operated by compressed air.

Electric Searchlight.—The electric searchlight, used in examining the roof of the tunnel before the lining had been placed, was made by the Rushmore Dynamo Works, being described as Type A, 110 volts, and 20 amperes.

Pumps.—In the tunnel, 6-in. Cameron suction pumps, operated by compressed air, were used to discharge the water into sumps, and from there the water was discharged into the sewers by electrically-operated centrifugal pumps.

"Battleships."—The "battleships", or buckets, were made of $\frac{1}{4}$-in. wrought iron, and were 4 ft. 8 in. by 7 ft. by 3 ft. 10 in. Their capacity was 4.6 cu. yd., water measure.

Drills and Columns.—Ingersoll-Rand 6-in. columns were used to drill the headings. The tripod drills used were generally Ingersoll-Rand, Type E-44 (with $3\frac{1}{4}$-in. cylinder), F-94 ($3\frac{5}{8}$-in. cylinder), and E-33 ($3\frac{1}{4}$-in. cylinder). A depth of about 2 ft. was drilled with each sharpening of the steel.

The extensive use of the tripod drills was probably due to the fact that, when this job started, the hammer type then on the market was not very efficient. The maximum length of steel that could be used with such drills at that time was 6 ft. Toward the close of the work, several new kinds of rotary hand-drills were put on the market, and these proved so satisfactory that they practically supplanted the heavier drills formerly used in this work. The first hand-drills used were the McKiernan-Terry non-rotating hand-drill, Style B. These were replaced later by the Ingersoll-Rand rotating hand drills, Type BCR No. 33 and BCR No. 430. The superiority of these small machines over the heavy tripod drills is shown by the fact

that they will drill, on an average, about 30 ft. per hour, and are now supplied with steels up to 12 ft. in length. The saving in labor and power has contributed to the success of these small drills. An ordinary tripod drill with a $3\frac{5}{8}$-in. cylinder requires, at 90 lb. pressure, 159 ft. of air, and must have a drill runner and a driller's helper to run it. The BCR No. 33 hand-drill, at the same pressure, requires 90 ft. of air, and can be run by a driller without a helper. The tripod drills require the steel to be changed every 2 ft.; but, with the BCR drill, from 6 to 8 ft. can be drilled without a change.

Drill Steel.—For the large drills, $1\frac{1}{4}$-in. octagonal steel was used; and for the small drills, $\frac{7}{8}$-in. hollow hexagonal steel. A set of steels for the large drills was made up as follows:

Length of steel.	Diameter of steel.
2 ft. 9 in.	3 in.
4 " 0 "	$2\frac{1}{2}$ "
6 " 0 "	$2\frac{1}{4}$ "
8 " 0 "	2 "
10 " 0 "	$1\frac{3}{4}$ "
12 " 0 "	$1\frac{5}{8}$ "
14 " 0 "	$1\frac{1}{2}$ "

Cost Data.

The following figures may be considered fairly accurate, as representing the cost of the various classes of work indicated:

Cost of excavation per cubic yard in double-deck tunnels, Sections 10 and 11; sound rock not requiring timbering; mucking done by air shovel..... $5.00

Cost of tunnel excavation per cubic yard in sound rock tunnel, no timbering being used (two-track tunnel, Sections 8 and 9)..... $7.00

Cost of tunnel excavation per cubic yard in unsound rock tunnels, requiring segmental timbering (two-track tunnels, Sections 8 and 9)..... $10.00

Cost of tunnel excavation in compressed air work (10 to 16 lb.). Only top of heading excavated. Segmental timbering used (two-track tunnels, Sections 8 and 9).... $15.00

In computing the foregoing figures, the following scale of wages and general charges were assumed, the scale of wages being based on an 8-hour day:

Superintendent	$300.00 per month.
Tunnel foreman	200.00 " "
Heading foreman	5.00 per day.
Mucking foreman	4.00 " "
Driller	3.68 " "
Driller's helper	2.00 " "
Cranesman	5.00 " "
Shovel runner	6.50 " "
Mucker	2.00 " "
Blacksmith	3.68 " "
Blacksmith's helper	2.00 " "
Electrician	4.80 " "
Electrician's helper	2.20 " "
Drill repairer	3.00 " "
Timekeeper	3.00 " "
Hoist runner	4.75 " "
Signalman	2.00 " "
Three-horse team	9.00 " "
Master mechanic	5.00 " "
Team checker	3.00 " "
Powder	$0.14 per lb.
Dump	0.30 per cu. yd.
Plant running and depreciation	1.42 per cu. yd.*
Administration charges.	10% of cost of labor and materials.

Concrete Lining.

The plans for the subway structure in the tunnel did not contemplate the placing of the concrete solid to rock in all cases. Where the structure had an arched roof, the side-walls were in all cases to be concreted solid to rock up to 2 ft. above the springing line of the arch. Where the rock was good and did not require any timbering during the progress of the excavation, the regular construction shown

* The figures for the Manhattan tunnels of the Pennsylvania Railroad are given in *Transactions*, Am. Soc. C. E., Vol. LXVIII, p. 192.

FIG. 28.—LONGITUDINAL CONCRETE WALLS OVER ROOF OF TUNNEL.

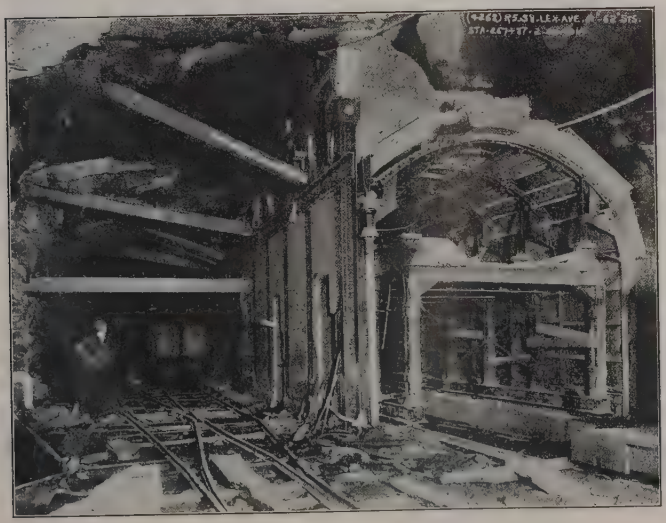


FIG. 29.—BLAW FORMS USED IN CONCRETING EXPRESS TUNNELS.

on the plans was put in, and a continuous concrete wall, 3 ft. thick, was built over the center wall, solid between the roof and the overlying rock. The space between the top of the side-wall and the center wall was then filled with hand-packing. In the tunnel, with four tracks on the same level, where the steel-bent construction was used, the structure was treated in the same manner, there being a longitudinal wall over each partition wall (Fig. 28).

Where the rock was unsound, and timbering had to be used for support while making the excavation, the structure was generally concreted solid to rock. To strengthen the timbering, the space between and back of the timber sets was first packed with concrete and grouted (Fig. 6), and, later, when the concrete lining was put in, it was placed solid against this concrete and the timber sets and the space between the timbers and the lining grouted. The grout used was generally a 1:1 mixture.

Mixing and Placing Concrete.—Central mixing plants were used on all the four sections. On Sections 8, 10, and 11, the concrete was mixed dry at the mixer, and water was added on the work; on Section 9, just enough water was used to wet the mass, and more was added on the work when considered necessary. The concrete was generally dumped on the roadway decking or on special platforms, and then transferred to the express tunnel below through iron chutes.

Several different methods of handling the concrete in the express tunnels were adopted on different parts of the work. The concrete cars were loaded directly from the chutes from the street surface. At first, the cars were dumped on a platform, and the concrete, after being passed by hand to several working platforms, was deposited in place. This crude method of handling concrete was soon abandoned, and various devices were used so that the concrete could be deposited directly on a working platform from which it could be shoveled into the forms.

On Section 8 (two-track tunnel), the common practice was to use a ramp leading from the floor of the express tunnel to a working platform. The cars were hauled to the platform, and the concrete was dumped and placed in the form by hand. Though this was an advance over the previous method, it was open to the serious objection that the ramps blocked the mucking tracks so that it was impossible to remove any muck while the concreting was being done.

LAYOUT OF CONTRACTOR'S PLANT FOR HANDLING CONCRETE
SECTION 9, EXPRESS TUNNEL

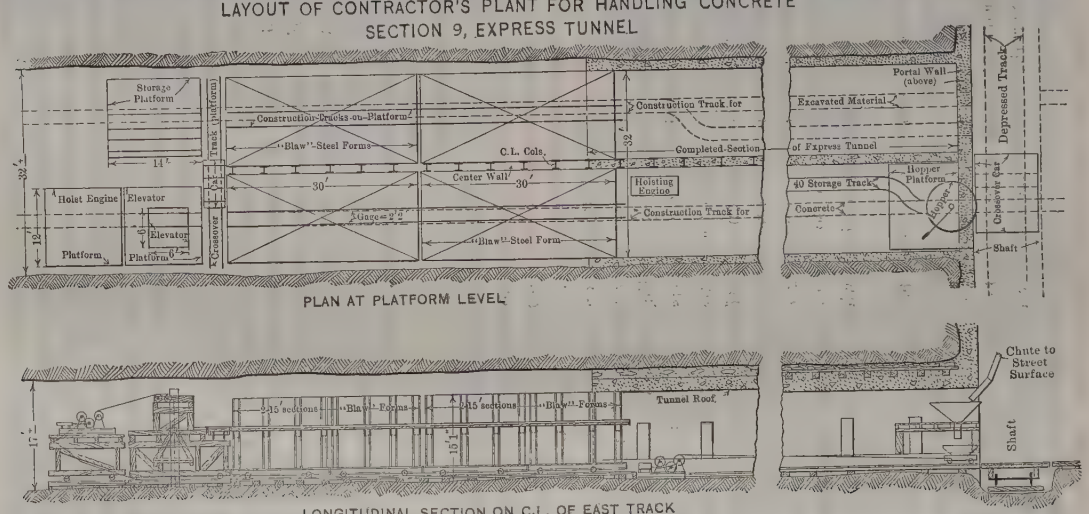
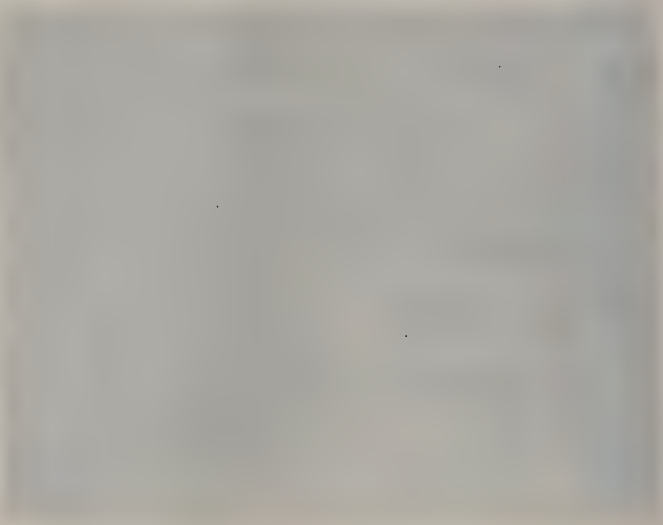
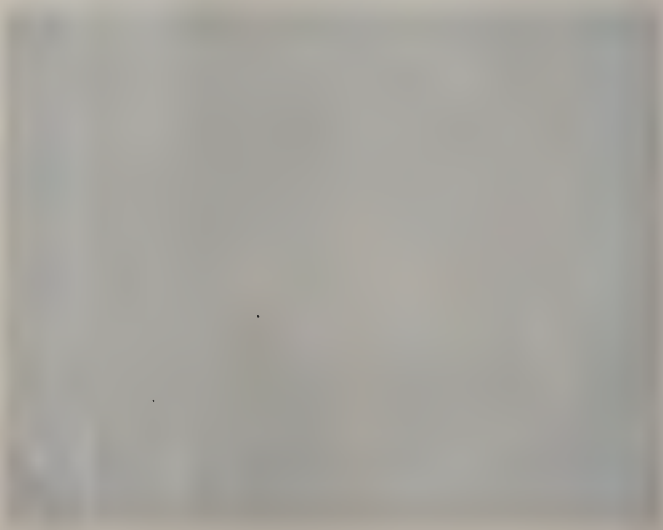


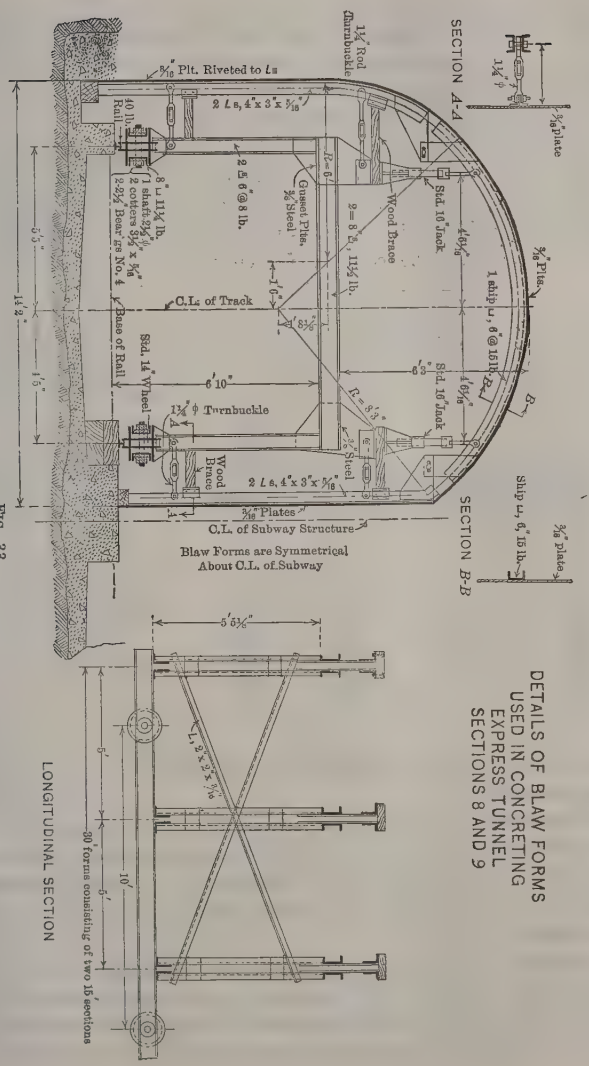
FIG. 30.

FIG. 31.—METHOD OF HANDLING CONCRETE IN TUNNEL, SECTION 11.

FIG. 32.—CONTINUOUS TRAVELER USED TO HANDLE CONCRETE IN TUNNEL, SECTION 11.







DETAILS OF BLAW FORMS USED IN CONCRETING EXPRESS TUNNEL SECTIONS 8 AND 9

FIG. 33.

Where this objectionable feature became so serious as to interfere materially with the progress of the work, a continuous working platform was built from the chute to that part of the tunnel which was being lined, so that there were separate tracks on different levels for both the mucking and the concrete cars.

To dispense with the necessity of a long working platform and double tracks, the contractor on Section 9 used a lift to raise the concrete cars to the level of the working platform (Fig. 30). The latter was provided with transfer tracks, so that only one lift was necessary to raise the concrete for either arch. This method of

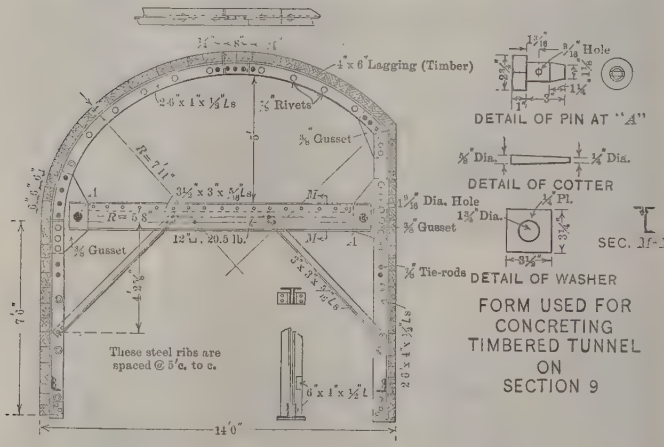


FIG. 34.

handling the concrete eliminated all the objectionable features previously mentioned.

Toward the close of the work, another satisfactory method of handling the concrete in the express levels was introduced on Section 11 (Fig. 31). The concrete car was mounted on a timber framework built on a flatcar, so that the concrete could be dumped directly on the working platform.

A continuous traveler or bucket elevator (Fig. 32) was tried for a short time, but was soon abandoned, as it did not prove very successful. The main objection was that the quantity of concrete that could be handled in this way was too small. The concrete for the

roof was deposited on the platform and then shoveled into the forms; the concrete for the side-walls was dumped in a sloping trough, leading into the forms.

Forms for Concreting.—Figs. 29 to 34 illustrate the principal types of forms used in concreting the express tunnels. Where the excavation was through sound rock which did not require any timbering, the general practice was to use the Blaw movable forms (Figs. 29 and 33). Where the tunnel excavation had to be timbered, the posts prevented the satisfactory use of the Blaw forms, it being impossible to move them. In these cases, forms of wood or those having steel ribs with longitudinal lagging were used (Fig. 34).

The tunnel work was started in April, 1912, and completed in December, 1914. The work was done under the supervision of the Public Service Commission for the First District, Alfred Craven, M. Am. Soc. C. E., Chief Engineer. John H. Madden, Assoc. M. Am. Soc. C. E., was in charge of the construction, as Assistant Division Engineer, until the end of March, 1914, at which time he was assigned to other work. The writer, who had acted as Section Engineer, succeeded Mr. Madden, and continued in charge until the work was completed.

DISCUSSION

Mr. Griest.

MAURICE GRIEST,* ESQ. (by letter).—This paper describes the construction of the rock tunnels in Lexington Avenue from 57th Street to 102d Street. It has been prepared, however, from the point of view of the construction engineer, rather than that of the designer, therefore some special features of the design will be presented, particularly of that portion from 97th Street to 102d Street, where the structure changes from a double-deck tunnel to a four-track tunnel on one level.

Two features seem worthy of brief mention: First, the flat-roof construction with transverse beams and concrete jack-arches and the reasons for its use; and second, the method of reinforcing the structure after the steel therefor was delivered, in order to provide for materially heavier loads than were contemplated originally.

The original design for this portion contemplated the use of concrete arches—twin tubes between 97th and 100th Streets as the local tracks spread out, and four single-track tubes from 100th to 102d Street as the grades approach. In the former case, the clear span varied from 14 to 30 ft.; in the latter case, the spans were uniformly 14 ft. The principal reason for adopting the arch form of construction in preference to the ordinary subway construction, was the greater ease in placing the concrete in the arch form than in the flat roof with smaller clearance above.

It was soon apparent that it would be desirable, on account of the condition of the rock, to construct one track at a time. To do so with the original construction, it would have been necessary to leave the centers in place in one arch until the adjacent arch was poured and packed to rock. Therefore, the ordinary flat roof construction with transverse beams, 5-ft. centers, was substituted, the steelwork being arranged to permit the construction of the center express tracks in advance of the excavation for or construction of the local tracks.

Between 100th and 102d Streets, the roof in general for the ordinary 14-ft. width consisted of a 20-in., 80-lb. beam supported between the tracks by 6-in. plate and angle columns, and at the sides by 12-in., 31½-lb. wall beams, designed for both roof load and side-wall pressure. This roof could carry safely the weight of 12 ft. of rock above it. It was expected that the rock would be sound enough to arch over, so that in no case would there be a weight greater than the foregoing carried by the roof. It was necessary, later, to increase the strength of the roof, for two reasons: first, at 102d Street, on account of the poor rock, open-cut excavation was extended beyond the original portal location, so that the maximum depth of fill over the roof was increased to depths varying from 28 to 38 ft. The roof, therefore, designed to carry 12 ft. of rock, was now to support from 28 to 38 ft. of back-fill.

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Mr. Griest.

As described in the paper, the rock in some cases broke out 12 ft. beyond the neat line of excavation, the excess excavation being refilled with hand-packed stone or concrete. In other cases, the rock was poor and seamy, so that the weight to be supported on the roof was problematic. It was necessary, therefore, to make provision for the support of more than 12 ft. of rock for a considerable part of the tunnel.

To meet these conditions, it was necessary to add from 50 to 100% to the strength of the roof. The steel roof-beams were already delivered. Several months would have been required to obtain additional structural steel. The conditions of the rock made it desirable, even imperative, that the permanent construction be placed without delay. The contractor had on hand a supply of rods ordered for other parts of the section. Furthermore, the rock had broken out, as described in the paper, above the neat line of excavation, so that the thickness of the roof could be increased without additional excavation. It was decided, therefore, to omit the jack-arches and increase the thickness of the concrete of the roof, so as to develop in effect a reinforced concrete slab.

The original plans showed an average thickness of concrete 3 in. above the top of the beams. This was increased to 12, 14, and 20 in. in various cases (making the depth of concrete 32, 34, and 40 in., respectively). Four 1-in. tension rods were added in each 5-ft. bay between adjacent bents. Assuming the roof to be a reinforced concrete slab, the reinforcing consisting of the rods and beams, the neutral axis is found to be above the top of the beam. The web of the beam, therefore, assisted in resisting the vertical shear, but did not resist any part of the horizontal shear at the neutral axis. Six ¾-in. inclined shear rods were added, therefore, in each 5-ft. bay over each support. By this method of reinforcing, the supporting power of the roof was increased 50% for 32 in. thickness, 70% for 34 in. thickness, and 110% for 40 in. thickness of concrete.

It was also necessary to reinforce the center walls to carry a load even greater than the supporting power of the roof. (The rock being caught up near or over the center walls and the space filled later with hand-packed stone and rubble, made it probable that a greater loading would be carried on the walls than on the intervening roof beams.) The columns, therefore, were embedded in 12-in. and, in some cases, in 16-in. concrete walls, with eight 1-in. vertical rods in each 5-ft. bay. The load was then assumed to be distributed as in a reinforced concrete wall, over the steel and concrete.

Mr. Madden.

JOHN H. MADDEN,* ASSOC. M. AM. SOC. C. E.—This work was of such magnitude, and the problems presented were so numerous, that the details would readily afford a field for several papers of the extent

* New York City.

Mr. Madden.

of that offered. Under such circumstances, the wisdom of illustrating the text profusely is apparent. The speaker believes that it may be of interest to review the development of the methods adopted for the tunneling operations and the experience during their execution, and, to that end, will devote his remarks in general to the work in progress for the first two years of these contracts, when he was in direct charge of the tunnel construction for the Public Service Commission. The author succeeded to that capacity in April, 1914.

Design.—For subways to carry intraurban traffic it is essential to provide ready access to stations, so as to offer the greatest convenience to the traveling public, and, where the stations occur at frequent intervals, as is necessary in New York City, this demands the minimum depth below the street surface consistent with a feasible re-adjustment of the existing sub-surface structures which will be interfered with. In the work under discussion, this was complicated further by the location of the express stations of the lower level, at which points a practical connection had to be provided with the local or high-level service.

With these considerations in mind, the small rock cover in places, with its incidental construction difficulties in the tunneling, can be readily understood. Future building operations or other sub-surface reconstruction involving a disturbance of existing conditions, was an added factor in the problem of the design of the structure.

Excavation.—The general character of the material encountered in the excavation has been sufficiently described, except that in many places a considerable stratum of destroyed rock, overlying the ledge rock, was penetrated. These deposits had been deprived of their cementing properties, and could usually be removed with a pick and shovel, though, when exposed in the face, they presented every appearance of ledge rock.

On starting the work, there was discussion as to adopting a bottom heading, but, in view of the many faults and seams common to rock of the character of Manhattan schist, it was feared that the use of this method might loosen the overlying material and complicate the subsequent removal of the upper section of the tunnel. With the top heading, support could be provided with the advance, before the ground had an opportunity to work and become heavy.

As shown in the paper, the finished two-track section was about 32 ft. wide, and the roof of the excavation was very flat. To permit the structure to be completed piecemeal, steel columns and longitudinal girders were substituted in the center wall for the original reinforced concrete design. By this means half of the structure could be built and blocked against lateral movement, and thereby avoid exposing the rock for the full section at one time.

Reference is made by the author to the completion of the upper level in advance of the work for the lower tunnels. It must be borne

in mind that it is necessary to maintain progress in work of this character, and that the contractor must base his methods on the preliminary information as to the character of the material which will be encountered. Except where it is known that the tunnel roof will project above the rock outcrop into a sandy soil continuing to the upper level, and thereby induce a bleeding off below that level, with the probable disturbance of the intervening material, extending to either side and entailing settlement outside of the normal limits of the work, it would seem that some advantage can be claimed for this procedure. The completion of the upper level excavation removes the overhead load if poor ground is developed in the tunnel, or if the rock cover above the roof of the tunnel is very thin. The timbering of the upper level excavation may also act to localize any disturbance caused by the tunnel excavation, by retaining the side banks above and diminishing the possibility of an extensive outside settlement. Mr. Madden.

Where apprehension is felt as to settlement from this sequence of operations, it is not believed that underpinning abutting buildings is as effective for their protection against injury as supporting their foundations on cribwork, as the latter admits of correction for settlement and, on the completion of the subway structure, such foundations can be extended to stable ground.

It is not intended, however, to question the wisdom of deferring the construction of the structure at the upper level in advance of the completion of the lower tunnel excavation where the latter is to be conducted through poor material. As stated by the author, where this course was followed, voids were caused under the upper structure, and, in some instances, a cavity was developed between the two levels. An attempt was made to close the neck of this crater with small timber, sealing above it with concrete and packing the sides of the cavity with sand bags to prevent sloughing off. Grout was introduced under pressure from below, in order to consolidate the disturbed ground, but, as no great success was attained in preventing further development of the settlement, it was necessary ultimately to reconstruct the footings and track floor. Indifferent results were likewise obtained from efforts to compact the soft ground underlying the upper level so as to retain it in place during the subsequent excavation from the lower tunnel.

Blasting.—The diagrams and data in the paper supply full details of the blasting operations. Where buildings abut on tunnel work and are founded on the same rock strata to be excavated, some concussion is unavoidable, but the damage from this source was remarkably slight, under the circumstances.

In the vicinity of some of the portals the breakage of windows was quite extensive, because of the air vibrations following the blasting, and though several expedients were tried to break up the velocity of the discharge at the portals, they met with no great success. The

Mr. Madden.

increase in the number of holes or the reduction in the charge of dynamite had no material effect in reducing the damage to the windows. The plate-glass store-fronts were protected with small wooden blocks placed on each side of the glass over which wires were carried to a frame at the top and bottom of the window. This operated to confine the oscillations induced by the vibration, and proved very efficacious in preventing the breaking or cracking of the glass. It was not practical, however, to place this device on each of the great number of small house windows, and, when broken, these were promptly replaced.

Emphasis might well be laid on the use of the powerful searchlight which was mounted on a car in the tunnel and used for periodic inspection of all exposed rock. This readily disclosed any indications of the rock working, or spalls loosening, and suitable precautions could be taken as required. In some instances where the ground was treacherous, the searchlight was used to examine the face of the heading following the blasting and before any men were permitted to enter. Its use was an insurance factor that is worthy of adoption on work of this character.

Timbering.—Where timbering was necessary, the usual segmental type was used, and the design contemplated either the support of the overhead load where the ground was heavy, or protection against injury through the dropping out of spalls or fragments where the rock was firm but blocky. When the soft water-bearing material was encountered at 74th Street, the contractor adopted the expedient (as mentioned by the author) of driving long spikes in the sides of the caps, and these projected from their face and formed a bond; the space between the timber sets was then packed solid with concrete through which the grout pipes extended. This method proved so advantageous that it was used subsequently in other parts of the work to which it would apply.

By this method the timbering was not only greatly reinforced, but a very efficient and uniform support was provided for the roof between the sets. A seal was also obtained for the water draining from above, and, where desirable, this could be collected and removed under control through grout pipes kept open for that purpose. In rock which is seamed and faulted, to the extent of that encountered in many places in this locality, it is important to minimize the time of the exposure of the face in the excavation, so as to reduce the tendency to work; and it is believed that concreting between sets as soon as practical after their erection offers an excellent solution of that problem. This is particularly applicable when the permanent lining cannot be completed coincident with the advance of the tunnel excavation.

As described in the paper, the work presented unusual problems in tunnel construction, and required from the contractors a severe tax on

their experience and resourcefulness. Acknowledgment is due to The Bradley Contracting Company and P. McGovern and Company for the successful completion of their respective sections. Mr. Madden.

ROBERT H. JACOBS,* ASSOC. M. AM. SOC. C. E.—This paper brings forcibly to the mind of the engineer the hazards necessarily incident to execution of work of this character. To those acquainted with such work, it is unnecessary to emphasize the fact that the process of driving tunnels, of the dimensions required for subway operation, in rock such as exists throughout Manhattan, within the limits of city streets, is somewhat hazardous, even when carried on under the strictest and most competent supervision. This applies not only to the force employed, but also to some extent to the public. It is felt, therefore, that it is a matter for congratulation that in the total of 9 700 lin. ft. of tunnel work, including 3 100 lin. ft. of four-track tunnel, covered by this paper, there were no accidents affecting the public, the only injuries suffered being slight damages to buildings due to vibration and concussion, and, in the case of soft ground tunnels, a possible slight settlement in a few buildings. Mr. Jacobs.

Considering the close proximity of buildings (Lexington Avenue being only 75 ft. wide), the character of the material excavated, and the millions of people passing daily over the temporary supports in the case of the cut-and-cover work, the record of casualties for the whole subway work is an enviable one. It may be stated, however, that the various processes of construction cannot be carried on with the same degree of safety as may be obtained in the operation of completed machines or structures. For example, if the same safeguards were thrown around the structural ironworker engaged on the erection of a skyscraper as are thrown around the passenger in the elevator of a completed building, the speaker is sure we would have no skyscrapers. In the same way, if the same safeguards were to be thrown around a tunnel excavator as around a subway passenger, there would be no subways.

It has seemed to the speaker that in undertaking this great work at the behest of our masters, the people, that the latter should be allowed to know in advance that there are certain necessary hazards incidental to such work, no matter how conscientiously and ably planned and executed. He is quite sure that such knowledge would in no wise deter them from their purpose to have these very necessary facilities provided. When, however, the public, through a lack of knowledge of the conditions under which work of this character is carried on, believes that it can and should be carried forward with absolute safety by the exercise of even ordinary care and skill, those in responsible relation to the work are subjected to the most severe criticism or worse, in the case of casualties.

* New York City.

Mr. Jacobs. This is being brought very forcibly to mind at this time by the recent failure of a portion of the erecting appliance of the Quebec Bridge. The general public is quite aware of the fact that the doctor is dealing with certain unknown elements in his work, and therefore does not hold him to strict accountability for the success of his treatment. It does not realize, however, that (although to a very much less extent) there are certain unknown elements which confront the engineer in the prosecution of work, which make it impossible for him to insure that it will be conducted with absolute safety, notwithstanding the fact that he is willing to accept full responsibility for the finished product. The lack of realization of this results in the engineer being held to an accountability which is unfair to the Profession. Here is an opportunity for educational work which can be accomplished by the engineer alone.

Mr. Thomson. T. Kennard Thomson,* M. Am. Soc. C. E.—Mr. Werbin is entitled to hearty thanks for his excellent presentation of a very interesting subject.

The speaker cannot help contrasting the difference, in many ways, between the construction of a rock-and-mud tunnel, on which he was engaged 32 years ago, and this up-to-date subway. This is exemplified by the difference between a small tallow candle and a large electric searchlight.

The success of the blasting operations on this work might also be contrasted with a case in New York Harbor some 15 years ago, when a coffer-dam was built as far out as it was considered economical, and the rock excavated inside the coffer-dam in the dry. From the finished excavated space (inside the coffer-dam), some fifty holes were drilled into the rock to be removed, outside the coffer-dam lines.

To ensure success the contractor secured a man from the explosives manufacturer to load the holes and connect the exploders in such a manner as to make a complete circuit. As the current would have to travel through this circuit, it seemed impossible to have a hitch; but only three or four exploders (at different parts of the circuit of fifty) were set off at the first attempt. The wires were then connected again, and a number, but not all, of the remaining charges went off. This naturally put the rest of the work in a very uncertain condition.

In order to ascertain the cause of the failure, the speaker took twenty-five exploders from the same stock, connected them up in the same manner as before, but without the dynamite, threw them into the water, turned on the current, and only four (at intervals) exploded. The remaining twenty-one were again connected up, and all went off together, showing that some of them were just a trifle quicker than the others.

* New York City.

Mr. Thomson.

The author speaks of the considerable quantity of extra rock excavation which the contractor was obliged to do without compensation. This is not equitable, but it seems to be difficult to draw up a contract which will allow for these changes; and, on public work, very few engineers dare to be just, against the terms of the contract and specifications, as they would be accused of graft, etc.

This refusal of the authorities to give the engineers a chance to use their own judgment, however, is costing the country millions of dollars, as the contractors have to bid higher than they would otherwise. As an example, an engineer who, in his time, had a reputation of the very highest, and did an enormous amount of work, got the reputation of refusing to allow any change in his plans and insisting on refinements which did not improve the job. The result was that his clients paid far more for what they got than they would have done otherwise. For instance, the manager of a large manufacturing concern once told the speaker that he always figured work for that engineer just as he would for anybody else and added 25% to the price to cover such arbitrariness.

Mr. Myers.

JOHN H. MYERS,* ASSOC. M. AM. SOC. C. E.—The speaker thinks that engineering literature contains many references to tunnels which approximate the size of the two-track sections described in this paper, and that he is correct when he says that there are few, if any, such references to tunnels similar to the four-track sections. He refers especially to the tunnel of varying cross-section by which is accomplished the transition from double-deck (or two tracks over two) to four tracks abreast between 98th and 102d Streets. He thinks, therefore, that Mr. Werbin has done a valuable service in compiling this accurate and complete record of this work.

There is one theme that runs all through the paper which should be emphasized, and that is the character of the design which permitted what may be termed sectional construction. By this is meant the design which allowed the tunnel lining to follow quickly after the excavation and enabled the constructor to excavate and line part of the tunnel before excavating and lining the remainder. The methods of construction which such a design made possible are illustrated in Figs. 1, 15, 16, 17, 18, 25, and 29, and Plate VII. It is the speaker's opinion that this design aided materially in the successful completion of this very unusual work through a portion of the city which presented many natural difficulties.

The fact that a structure of the magnitude of that described has been built through the heart of the city without material interruption of its mode of living, or of its varied industries, is in itself worthy of note, and the record of the methods used in bringing this about, which the paper provides, is, the speaker believes, of permanent technical and historical value.

* New York City.

Mr. Moulton.

H. G. Moulton,* Esq.—The tunnels on Lexington Avenue, especially the double-deck sections, approximately 40 ft. high and 40 ft. wide, are quite unusual in size, from a civil engineering standpoint; yet they are small in comparison with underground excavations required in mining operations. The work on these sections presents problems more nearly comparable to metal mining than those encountered anywhere else on subway construction in New York City.

In mining large bodies of ore, it is often necessary to carry a stope, for a width in excess of 100 ft., to a height of 100 ft. (the usual distance between levels), and for a length of several hundred feet on the strike of the vein, and to carry the walls, roof, and face of the excavation on timbers. As the work on Lexington Avenue is so similar in type, though on a much smaller scale, it is interesting to consider the possibility of adapting to work of this kind the methods used in extracting irregular ore bodies, the dimensions of which are too large to permit the walls and roof to stand for any length of time without support.

There are two marked differences between the excavation methods described in this paper and those which would have been used under metal-mining practice. The first difference is in the method of attack—the use of the top heading instead of a bottom heading, as in mining operations. The second is in the method of timbering the opening—I-beams, segmental sets, etc.

In the very early days of western mining development, it was common practice to remove ore bodies in very much the same manner as that described in this paper. The method is known as underhand stoping, and consists of carrying the upper level of the excavation in advance of the lower, and following in successive levels with the excavation of the bench. As mining methods developed, it was realized that this was the least economical way of attack, and it was abandoned in favor of overhand stoping or, in other words, bottom heading work, in which the lowest part of the excavation is carried well in advance and the back is taken down in successive slices, stepping back consecutively toward the roof. This method has the advantage of less shoveling and also of cheaper breaking, as the rock is broken from an under-cut horizontal face at less cost than from any other position, and also because the drilling can be done with the most efficient types of drills—the so-called stope hammers or buzzers.

In order to hold the walls and roof, and keep the men close to the working face, timbering is placed in sections as the work progresses. The usual dimensions of the timber sets common in metal-mining practice are 7 ft. high, 5 ft. wide between posts, and 5 ft. longitudinally between bents or adjacent rows of posts. The posts, caps, and girts are all framed to exact lengths and provided with horns in daps, so that connection is secured for each member of the set in every direction.

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The rock is broken out in small sections, say, 5 ft. square and 7 ft. high, and another set of timbering is placed. On all slices above the bottom heading, the new post rests directly on top of one of the posts set in the bottom heading, and the cap and girt join with the set at the side and to the rear. In this way the entire mass of rock is removed in a series of inverted steps in such a manner that not more than 5 ft. in either direction is without timber support at any time. The probability of an extensive rock fall is eliminated, and it is possible to work exactly to the pay line of the excavation.

To apply square-set stoping to such an operation as that described in this paper, the first step would be to sink a shaft at one side of the excavation and drive a bottom heading in each direction in the lower corner of the section nearest to the shaft. This bottom heading would be continued through to the next shaft and timbered continuously with the foundation or sill floor timbers of the square-set system. At different places along the heading, the stopes would be started, widened to the opposite side of the section, carried up to the roof, and then continued in both directions until several hundred feet had been opened and timbered. After this, a start would be made on steel erection, the timber sets being pulled out row after row, after alternate bays of steel had been erected and concreted between them. Where no heavy pressures had been developed, the sets would be removed quite readily after knocking out the side and top blocking and wedging. Where heavy pressures had developed from loose ground, it would be necessary to do considerable work with axe and saw on the bottom posts, and ruin part of the timber. Yet, even under the worst circumstances, it is possible that 75% of the timbers could be recovered and used over and over again as the work progressed.

As to the cost of this method, ore is mined on an extensive scale in various districts at costs ranging from $1.50 to $2.50 per ton by square-set stoping at reasonable depths, exclusive of any items corresponding to cost of plant or contractors' profits. This would be equivalent to about $3 to $5 per yd. The timber required would be approximately, 30 ft. b. m. per cu. yd. of material taken out, and, with timber at $60 per ft. b. m. in place, the cost would amount to $1.80 per cu. yd. Considering that most of the timber would be used over and over again, and also that round timbers would be the most advantageous to use, and much less expensive than sawed stuff, it is probable that $1 per cu. yd. would represent the maximum cost for timbering. Against this additional cost there would be certain credits, as compared with carrying the excavation open and with no timbering at all. One such credit would be in cheaper breaking and mucking costs, due to shooting the rock down from an under-cut face and catching it on heaving blocking of poles resting on the caps of the timber sets. It could then be shoveled

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into chutes and drawn into cars in the main bottom heading. Another saving in cost would be realized by working exactly to the pay line, and saving the excess cost of hauling out irregular masses of material, broken away outside of the section, and afterward concreting and backfilling these spaces. In blocky ground, such as was often encountered on Lexington Avenue, it is probable that the credits from the items mentioned would more than offset the additional cost of $1 per cu. yd. for timbering, so that this method might well be cheaper than the ones used.

As to the question of safety, there can be little doubt that full square-set timbering would have the advantage. The great principle involved in keeping rock excavation safe is to have the smallest possible quantity of ground standing without support. On Lexington Avenue, this was done by building the finished structure in sections; but the same thing could have been accomplished, and even greater protection secured, by putting in the timber in small sections and having only the extent of one set open at any one time. Then, the steel structure could be built in its entirety between the timbers at any distance desired behind the face.

As to speed, all the advantages are with the bottom-heading method of attack, as it is possible to open up as many faces as desired dependent only on the hoisting capacity of the shafts.

It is well recognized that even the worst ground may be held with comparative ease before any movement starts, but if once it starts, it may demolish any timbers that can be put in. Close timbering, to prevent the initial slip, may mean all the difference between absolute safety and serious collapse. The wedging and arching action of blocks of rock in unsound ground, together with the frequent resistance on seams, and probably to a certain extent atmospheric pressure, renders valuable assistance to the timbers in holding the ground, and the timber system should provide for this by preventing the initial movement which may destroy all these valuable allies.

As to the timbering methods used in unsound ground on Lexington Avenue, and described in this paper, the speaker is of the opinion that although they served their purpose in giving protection while the work was being carried on, nevertheless, they were unnecessarily expensive and inefficient, as compared with the usual mine-timbering methods. The segmental set, for example, which is used so widely in large tunnel headings, represents the most inefficient and uneconomical manner in which timber can be used. It usually comes down at the joints long before the timber can develop its strength as an arch, and wherever heavy ground is encountered in the roof, the segmental sets must be reinforced with longitudinal horseheads under the top cap and radial posts at the joints. About all that segmental sets will accomplish is the

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prevention of spalling and light rock falls from the roof. Where really heavy ground is encountered, as in extracting ore by the caving system, very different methods of timbering are used. Good examples of this are to be seen in some of the leading mines in Arizona, where the ore is extracted by honeycombing the lower part of a block with tunnels and then shooting out the adjacent supporting pillars, so that the entire block drops 6 or 7 ft. and is crushed by its own weight. To keep the main haulage drifts open below such a block of ground, they are timbered with long flat caps reinforced with a short cap in the center, blocked against knee-braces resting on short posts within the main posts of the set. This type of set is so much superior to any attempts to obtain arching action through segmental timbers that it is surprising it is not generally adopted in timbering railroad tunnel headings through heavy ground.

There is a general principle involved in timbering rock excavations, namely, that small posts acting in compression form the most economical method of support. Mine timbering should be placed in small units, and the use of timber in beams or in arches should be avoided as far as possible. There is also a great advantage in smaller units, in that they are easily placed and with fewer men.

The use of many of the methods shown in the paper, such as long I-beams, cribbing built up to support long beams and packing above, is reminiscent of descriptions of the early attempts of metal miners to support ground prior to the development of square-set stoping. The largest timber available was often taken into the mine, and great posts were built up by splicing, perhaps to 60 ft. or more in length. Some of the stopes in the old Elkhorn Mine, in Montana, furnish a good example of these massive long stulls and posts. It requires many times the quantity of timber to work in this way that square-set stoping in small units would call for. Furthermore, the work is not as safe, and the labor required in handling these great units is excessive, as compared with that needed to place the smaller timbers. The same argument applies to the use of I-beams in underground work. Had the miners of a generation ago been able to obtain 24-in. and 36-in. I-beams, they would probably have stuck some of the stopes full of them in desperate attempts to span wide openings and carry the roof without posting support, but they would have soon abandoned them as they did their attempts to splice and build up long posts and beams.

The time required with the mining methods described would depend entirely on the number of faces opened. In the Butte District, which is one of the best examples of square-set stoping in mining operations, well in excess of 10 000 cu. yd. of material are removed daily, from depths ranging from 2 000 to 2 800 ft. below the surface, and the daily extraction is varied in strict accordance with the ore requirements of the smelters, by increasing or decreasing the number of men at work on

Mr. Moulton.

the various stoping faces. If such a method were applied on a job like the Lexington Avenue Subway, it would be merely a matter of carrying the bottom headings clear through and attacking at as many places as desired. The work could be speeded up or slowed down by opening more faces and operating more drills, or by shutting down on different faces.

The whole subject, of course, is quite academic at the present time, because the Lexington Avenue Subway is finished and practically ready for operation. The work there was splendidly done, and the completion of the finished structure in its present form, under the difficult conditions encountered, was a feat for which the Bradley Contracting Company and the Engineering Department of the Public Service Commission justly deserve the fullest measure of congratulation and appreciation. Nevertheless, Lexington Avenue was about the only place on the New York subway construction where metal-mining methods might have been applied extensively with advantage, and this paper, therefore, makes pertinent a general discussion as to comparisons of such methods with those which the contractors adopted.

Mr. Ridgway.

ROBERT RIDGWAY,* M. AM. SOC. C. E.—It would seem from a question asked during the discussion of this paper that the author has not made plain the practice followed in loading and firing the heading holes. Only those holes were loaded which were to be fired in one shot. This was also the practice in the headings of the City Aqueduct Tunnel of the Catskill Aqueduct, and is far safer than the method formerly used on some works in the vicinity of New York City, which was to load the whole face before firing any of the holes.

The author has referred to the presence of streams crossing the line of Lexington Avenue at 57th Street and at 75th Street, where soft ground was encountered in the tunnel headings. These streams, of course, indicated valleys in the surface of the country before the streets were graded. These and similar streams along the line of the work added greatly to the difficulties of construction. In some cases, the flow of these streams had been taken care of by roughly built stone culverts, but, in other instances, the street grading had been done without making any provision for the flow, and the water was allowed to flow through the rock back-fill, eventually finding its way into the sewers, or the East River. Relative to this matter, attention is called to the value of the topographical map of Manhattan Island, published in 1874 by Gen. Egbert L. Vielé, as an aid to those doing sub-surface work. This map, popularly known as the "Vielé map", shows the old watercourses and swamps quite accurately, and whenever particularly bad ground was encountered in subway work, the reason for it was usually indicated on this map. An engineer engaged on

* New York City.

foundation work in New York City told the speaker that whenever he is called on to make a special investigation and report on such work, he looks at the "Vielé map" to ascertain whether or not there was an old stream or swamp near the foundation in question. Mr. Ridgway.

The problem to be met by the construction engineers in the work described was a most complicated and difficult one, as a reading of the paper shows. The speaker doubts whether a tunnel more difficult to construct has ever been built in a narrow city street than the one between 99th and 102d Streets. That it was completed successfully, without serious accident of any kind, is cause for congratulation, and indicates the care with which it was supervised. The speaker has an intimate knowledge of this work and is glad to testify to the faithful and intelligent direction of it by the Division Engineer in charge, John H. Myers, Assoc. M. Am. Soc. C. E.; the Assistant Division Engineers, John H. Madden and I. V. Werbin, Associate Members, Am. Soc. C. E.; and the Assistant Engineers in charge of the several sections of construction. Under Mr. Madden the work was begun and well advanced. When he was transferred to other work, he was succeeded by Mr. Werbin, who, up to that time, had been Assistant Engineer on one of the four sections. Mr. Werbin is well fitted to describe the work, on account of his close connection with it and the very careful attention which he has given to all its details of construction.

Much credit is also due the Bradley Contracting Company, the Contractor for Sections 8, 10, and 11, and P. McGovern and Company, the Contractor for Section 9.

There was a large excess of excavation outside of the tunnel lines on the sides and top, and, in order to protect the work and prevent loose rock from coming down, the thickness of the concrete was increased and the walls thus strengthened. Some of the contractors expect to receive compensation for this extra concrete, but none has yet been paid.

C. V. V. Powers,* M. Am. Soc. C. E.—Referring to Mr. Ridgway's remarks about the Vielé map: In the course of some investigation at the office of the Bureau of Street Openings, the speaker found original maps and surveys much older than the Vielé map. They are interesting and valuable as giving first-hand information in very considerable detail concerning old surface conditions. Any one desiring to consult them can no doubt do so by making proper application. Much of the information contained in the Vielé map was probably taken and condensed from these earlier drawings. Mr. Powers.

Francis Donaldson,* Esq. (by letter).—The writer has read with interest Mr. Werbin's paper describing the construction of the Lexington Avenue Subway tunnels, and Mr. Moulton's discussion of it. He has also noted Mr. Moulton's conclusion that the work could have Mr. Donaldson.

* New York City.

Mr. Donaldson.

been performed more economically and safely by mining with square-set timbers than by the timbering methods actually used. The writer has driven tunnels with both top and bottom headings, is familiar with mining practice, as well as with the Lexington Avenue work, and yet is unable to agree with Mr. Moulton, whose conclusion is too sweeping, to say the least.

In the first place, the choice of a method in any kind of construction must be governed by the results desired. In ore mining, the requirement is the excavation of the maximum quantity of material at the lowest cost consistent with safety, whereas, in driving a tunnel beneath a New York City street, it is not so important to excavate material at the lowest cost per ton hoisted as to excavate it in such a way as to prevent the settlement of the surrounding ground and facilitate the construction of the permanent lining. Secondly, the successful application of any method depends largely on local conditions, and a plan that might be best in California, where timber is cheap and intelligent miners plentiful, might not work at all in New York City.

To build the subway tunnels from 53d to 106th Street was a big job, and it had to be done in a limited time. It was necessary to adopt methods which would enable the kind of labor available in the vicinity to be used to advantage. New York City was full of good tunnel men of all kinds, from superintendents to muckers, who had learned their business on the many aqueduct and subway tunnels recently driven in and around the city and in near-by States, but there were few miners, in the western sense. If mining methods had been required by the contract, it would have been a practical impossibility to build up an organization of miners large enough to complete the work in the required time. That tunneling methods were proper, when viewed from this standpoint, is shown by the fact that the job was completed on time and in a satisfactory manner.

To consider more specifically the different types of tunnel structures described in the paper: The express tracks in Sections 8 and 9, between 53d and 79th Streets, were in a tunnel very nearly equivalent to a standard double-track railway tunnel, driven for the most part in solid rock, and lined with a concrete arch. It is generally believed by American tunnel engineers—certainly by the writer—that the top heading and bench method is the cheapest and best for tunnels of this type, and that, where timbering is necessary, segmental timber fills the bill.

Examples of the successful use of segmental timbering in tunnels through all kinds of ground are so frequent that it is hardly worth while to refer to them; but one recent instance refutes Mr. Moulton's condemnation so emphatically that it deserves mention. A double-track railroad tunnel was driven by the Northern Pacific Railroad near Tacoma, Wash., in 1912 and 1913, through heavy clay and sand

wet enough in places to have "a tendency to flow around the breast boards into the heading".* This tunnel was excavated behind a roof shield, and was timbered with segmental sets of 12 by 12-in. timbers, placed on 2-ft. centers, and lagged on the outside. Each set consisted of eleven segments. The lining proved entirely stable, and required less than 20 ft. b. m. per cu. yd. of material removed. Mr. Donaldson.

To return to Sections 8 and 9: it seems certain that, where a tunnel can be driven safely without timber, it is not good engineering to timber it. Segmental timber can be placed in short stretches as the excavation progresses, or after the excavation is completed if it seems necessary, and when placed it offers no obstacle to the construction of the permanent arch ring. Square-set timbering, on the other hand, must be placed as the excavation is removed, regardless of the character of the rock; it must fill the entire volume excavated, and consequently it will interfere to the greatest conceivable degree with the arch centering and the building of the arch itself.

A further objection to square-set timbering is this: the Lexington Avenue tunnels were driven so close to the surface that, even where the borings showed good rock, there was a likelihood of cutting a fissure filled with running ground and extending to some point below the roof line of the tunnel. Now if it is supposed that a bottom heading misses the fissure and that the next lift reaches it, the bottom will be neatly stoped out from under, and a run will be started that will stop when the soft ground gives out or the tunnel fills up. For an example, refer to the accident that occurred in driving the Mont d'Or Tunnel, under the River Bief Rouge, in 1912. This danger can be avoided by a top heading properly timbered to secure the roof. If necessary, the use of compressed air can be resorted to, as was done at 56th and 57th Streets.

Sections 10 and 11, between 79th and 106th Streets, included two general types of construction. The stretch from 79th to 98th Street was for the most part a double-deck tunnel, two tracks wide, with a steel lower story supporting a concrete arch roof. The rock, as Mr. Werbin states, "was fairly good, and very little timbering was used". The arguments just cited, bearing on the proper construction methods for Sections 8 and 9, consequently apply with equal force here. Certainly, the conditions and methods of excavation illustrated by Figs. 11 and 13 are as nearly ideal as the most grasping contractor could desire.

From 98th Street to the end of Section 11, the tunnel varied in cross-section, but was characterized by a flat roof supported by steel bents on 5-ft. centers, with rows of columns between the tracks. This is a type of construction to which segmental timbering is entirely

* *Engineering Record*, February 7th and 27th, 1915.

Mr. Donaldson.

unsuited, and ordinary timbering methods can be used to advantage only if the rock is so solid that it will sustain itself without support over a flat roof at least as wide as a one-track structure. The contractor, nevertheless, attempted to continue the plan that had been successful on other parts of the work, and kept his air shovel working until unsound rock north of 100th Street stopped him. A heavy and elaborate system of timbering failed to prevent large over-breakage in the roof. It was considered cheaper to take down loose rock above the roof line than to hold it in place, and at one point 12 ft. of rock were removed. Large quantities of excess concrete and dry packing were required.

The writer believes that from 98th to 103d Street, square-set timbering could have been used advantageously in the manner that Mr. Moulton suggests, and that the contractor could have saved money by using it. The cost, however, would probably have exceeded the higher figure given by Mr. Moulton, namely, $5 per cu. yd., due partly to the difficulty of organizing the job in New York City, but chiefly to the fact that the volume of rock removed in drifts and cross-cuts would have been much greater in proportion to the total volume than is the case in mining. Mine slopes are comparatively high and narrow, but these flat-roofed tunnels were low and broad.

Mr. Werbin.

Israel V. Werbin,* Assoc. M. Am. Soc. C. E. (by letter).—Mr. Moulton's discussion has perhaps set many to thinking of the possibility of adapting mining methods to excavating tunnels for engineering construction. The suggestions made by him are undoubtedly interesting, but, as one who has been intimately connected with the work described, the writer must take exception to his conclusion that mining methods could have been used advantageously on certain parts of the work.

Referring to the large double-deck tunnels, with a cross-section approximately 40 ft. square, Mr. Moulton states:

"The work on these sections presents problems more nearly comparable to metal mining than those encountered anywhere else on subway construction in New York City."

The mistake is made right at the start in assuming that there is any similarity between metal mining and the work described in the paper. There were several important factors connected with the work in question which eliminated any possibility of using any of the methods suggested by Mr. Moulton, and these were:

(1) That the work was done under narrow streets, in built-up sections of the largest city in the world;

(2) That the tunnels were driven dangerously near the rock line;

* New York City.

(3) That the rock was known to be of a dangerous character; and
(4) That the problem was not merely one of making an excavation, but of doing this so that there would be absolutely no settlement of the street surface, and, most important of all, of building a structure inside of the finished excavation. Mr. Werbin.

Surely none of these problems is to be met in mining work, particularly if credence is to be given to the occasional newspaper reports of large areas adjacent to mining operations settling, on account of the work done beneath them.

Mr. Moulton criticizes the use of the top heading as being antiquated and expensive. This manner of attack is followed almost everywhere in tunnel operations in America. This fact may not perhaps make it the better method, but there were other important considerations which made it the only practical method to use on work of this kind.

The tunnels in question were driven in connection with subway work, and, from the point of view of economy of construction, it was desirable to keep the structure as near to the street surface as possible. The importance of this feature will be realized when it is noted that the cost of the excavation in such work is approximately 50% of the cost of the entire construction. It is also important, from the standpoint of operation, for the railroad at the stations to be as near to the street surface as practicable, so as to reduce to a minimum the height of the stairs, and to dispense with the use of elevators.

When the first subway in New York was built, a portion on upper Broadway was built in a deep tunnel, and access to the stations at 168th, 181st, and 191st Streets is obtained by using elevators. Experience shows that with the large amount of traffic handled by the city railroads, the use of elevators is not only an expensive feature of operation, but is highly undesirable. For this reason, on the new work in the double-deck tunnels on Lexington Avenue, the express stations were placed immediately beneath the local tracks, so that communication with the local stations could be had by short stairways instead of elevators. In order to maintain reasonable grades, it was necessary to keep the express tunnels a short distance below the local tracks. If a bottom heading had been used, good rock (not requiring any timbering) would have been encountered all along the line, except for a stretch of about 200 ft. at 74th Street. There were, however, several places along Lexington Avenue where old watercourses ran across the line of the subway structure, and at these places the surface of the rock was considerably below the roof line. The material directly above the rock at these points was a fine quicksand or alluvial earth, which, when encountered, was very difficult to hold and keep from being washed into the excavation.

Mr. Werbin. Under such conditions, the use of a bottom heading would have been dangerous. The top heading had the advantage of acting in the nature of an exploration tunnel, so that the contractor was prepared in advance for the conditions with which he had to deal. When bad ground was encountered, it was taken care of by timbering, with or without lagging boards, as might be necessary; and, when the overlying material was once properly caught up and held, the element of danger was practically removed. Assuming that under such conditions the work was conducted with a bottom heading, it would mean that a very expensive system of timbering would have been necessary every time another lift was taken out, with the continuing possibility of the danger and damage that such an operation would have involved.

It must be borne in mind that the structure for the local tracks had been completed in advance of the express tunnel excavation, this being necessary in order to maintain necessary progress in the work. The loss of any material between the local structure and the express tunnels, therefore, would involve damage to the completed local track structure, in addition to settlement in the buildings at the street surface. It might also be mentioned that the contractor was held absolutely liable for all damage to buildings that resulted from his operations, irrespective of the matter of negligence, so that a procedure which would reduce to a minimum the possibility of damage might ultimately prove the most economical, even though some other method might permit the removal of the excavation more economically. When these matters are considered, the writer is sure that no mistake was made in using a top heading.

Mr. Moulton claims that with the methods of timbering ordinarily used in metal mining, it would be possible to work more nearly to the pay line of the excavation. The writer fails to see how the method of timbering can in any way affect the lines of excavation. This matter is determined more or less by the character of the rock and the way in which the strata lie. The writer believes that the men engaged on such works as that described in the paper are fully as competent as mining men to carry the excavation to approximate neat lines. The method of cribbing up the entire excavation might serve to hold some of the rock in the roof from falling down, but, as the writer has tried to bring out in the paper, no attempt was made to hold up such rock by timbering, the consensus of opinion being that the better practice would be to remove all loose rock. This conclusion was reached after careful study, there being little doubt that any timbering that would have to be put in to hold such rock in place, would be far more expensive than the additional cost of removing such excess excavation. The rock along the work in question had two seams running almost at right angles to each other, with the apex pointing upward, so that the rock in the roof was more or less wedge-shaped,

with the bottom of the wedge downward. The very dangerous condition is evident, and, when this rock persisted in working loose, even where timbering was placed, there was no doubt that it was good practice to remove these large masses of loose rock rather than attempt to hold them in place, where they would have been an ever-continuing menace to the lives of the men working in the tunnel. Mr. Werbin.

Mr. Moulton has criticized the segmental type of timbering as being expensive. The answer to this is that the problem was not only one of making an excavation, but also of building a structure inside the completed excavation. The square-set system of timbering used in mining, described by him, may be all right in a mine, but would make the cost of the construction that had to follow the excavation almost prohibitive. The subway structure could not have been erected without shifting considerable timber, and no engineer engaged on such work will deny that, once bad rock is caught up, it is very poor and dangerous practice to disturb the timbering.

The almost universal use of the segmental set in bad ground was prompted by the fact that, after it had been placed, the structure could be built beneath it with very little moving of the timbering. Another great advantage in the use of segmental timber in this work was the fact that, almost throughout the work, water was encountered in great quantities, and, by concreting between the timbers and grouting, a temporary roof was made. The water was then concentrated and led off through the grout pipes, and in this way was prevented from washing away the fresh concrete in the finished structure until it had had a chance to set.

The use of the longitudinal beams mentioned in the paper was not advocated so much on account of permitting any economy in removing the excavation, but solely because, with such a system, the posts directly under the sets did not have to be moved every time an additional lift was taken out; and it afforded a good foundation for the posts, irrespective of soil conditions. The beams and posting, therefore, could be placed where they would offer the minimum interference with the construction work that was to follow. The Lexington Avenue work was the first in which the large longitudinal beams were used to any extent, and the advantage of their use has been so marked that this system is being adopted on much of the more recent work.

Mr. Moulton has presented some data as to the estimated cost of making the excavation if mining methods had been adopted. He does not seem to have taken local conditions into consideration, and if, to the prices he gives, he adds the cost incurred by the contractor practically having no choice in the location and the number of shafts, the high cost that must be paid in the city for labor, plant sites, dumping privileges, superintendence and insurance costs, and the

Mr. Werbin.

increased cost due to the drilling and blasting methods required in a job of this kind, the total sum for doing the work would be found to be higher than he apparently thinks it would be.

The writer is sure that, if Mr. Moulton had been familiar with the conditions peculiar to this work, he would not have advocated the use of square-set timbering, or conducting the excavation with a bottom heading. It would seem to the writer that the only place where such a method could have been considered would have been in connection with the tunnels of 100th and 102d Streets, where the spans were greater than 60 ft., provided very bad ground had been encountered. The large I-beams to hold up the roof in this part of the work were used by the contractor mainly so that he could continue to utilize his air shovel, but, as stated in the paper, this method of procedure was found to be very expensive, and of little advantage, so that the use of the air shovel was abandoned and the mucking was done by hand.

It was found possible to excavate the section for sufficient width to permit the construction of one track of the structure at a time without using any timbering whatever, except for occasional posting; and, after the structure for this one track was completed and the roof had been caught up, the excavation for the adjacent track was made.

It would seem to the writer that the method followed was better than to attempt to make the excavation wider and use timbering. The only objection to the piecemeal method of construction was the damage to the finished structure that resulted from carrying on the blasting adjacent to it, but this could have been avoided had the contractor taken proper precautions.

It might have been advantageous to use the methods suggested by Mr. Moulton, provided the rock encountered would not have permitted the excavation for one track with practically no timbering.

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Paper No. 1389

A METHOD OF DETERMINING A REASONABLE SERVICE RATE FOR MUNICIPALLY OWNED PUBLIC UTILITIES*

By J. B. Lippincott, M. Am. Soc. C. E.

With Discussion by Messrs. Frank S. M. Harris, T. Kennard Thomson, W. B. Yereance, Leonard C. Jordan, H. F. Clark, Allen Hazen, W. G. Irving, H. A. Whitney, and J. B. Lippincott.

Synopsis.

The physical development of the Western States is far from complete, and although the larger municipalities may be able to finance utilities satisfactorily, in a manner commensurate with their needs, the growth, particularly of the outlying districts and smaller towns, will await largely the investment of private capital. In order to advance, all communities must have light, transportation, telephone service, water, etc. These utilities, whether privately or publicly owned, are essential to development. In California, at present, exclusive of steam railroads, 95% of the value of the public utilities is in private ownership.

In addition to the advisability of the public administration of both publicly and privately owned utilities being conducted on a fair and reasonable basis, it is essential that this administration should be such as will encourage the investment of private capital therein. If the rates for service of publicly owned utilities are too low, and

* Presented at the meeting of October 18th, 1916.

deficits are made up from the general tax funds, such administration is unfair to privately owned utilities, and will tend to discourage the investment of private funds therein, in States practicing this policy. Of late, and at present, in California, voluntary investment in privately owned utilities is awaiting the determination of public policy relative thereto.

The rates for service of a publicly owned utility should be fixed in the same manner as that for one privately owned, that is, it should be based on operation, plus depreciation, plus reasonable interest on value. Any surplus resulting from such administration should be deposited in the public treasury of the city to the credit of the general tax fund.

Public utilities are the framework on which the commonwealth is developed. All classes of citizens desire ready access to transportation, light, water, power, and telephones, in order to develop their communities economically. Excluding steam railways, 95% in value of the public utilities of California are privately owned. Statistical tables (Tables 1 and 2) show that not more than 10% of the State of California (after omitting the area of the desert lands south of Owens Lake) is developed as to density of population, and about 20% as to value. The wealth per capita is high. It must be borne in mind that the growth of a community, and its attendant wealth, are largely dependent on public utilities; and the investment of private capital in these enterprises should be fostered and encouraged with a view to the further growth of the State. A new and undeveloped State, such as California, manifestly cannot publicly build all its utilities.

There has been difficulty in obtaining the data on the relative value of municipally and privately owned utilities in California. That of the privately owned utilities for 1912 is taken from the report of the State Railroad Commission. This covered all the privately owned utilities, with the exception of the steam railroads, which have not been pro-rated for the different States. The best information on the municipally owned utilities is found in the report of the California State Controller for 1914. This report gives statistics relating to the municipal utilities in the State. For some of them, however, the values were not stated, and some were incomplete, as in the case of the Los Angeles Water Department, where the aqueduct was omitted. To

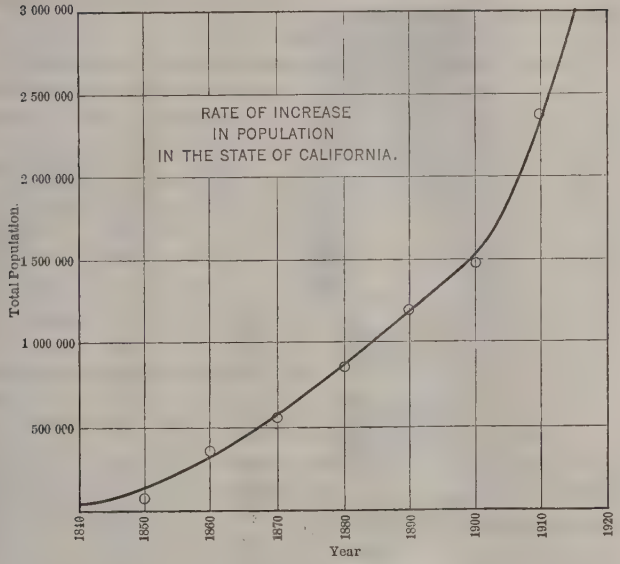
RATE OF INCREASE
IN POPULATION
IN THE STATE OF CALIFORNIA.
Total Population.
3 000 000
2 500 000
2 000 000
1 500 000
1 000 000
500 000
1840
1850
1860
1870
1880
1890
1900
1910
1920
Year

FIG. 1.

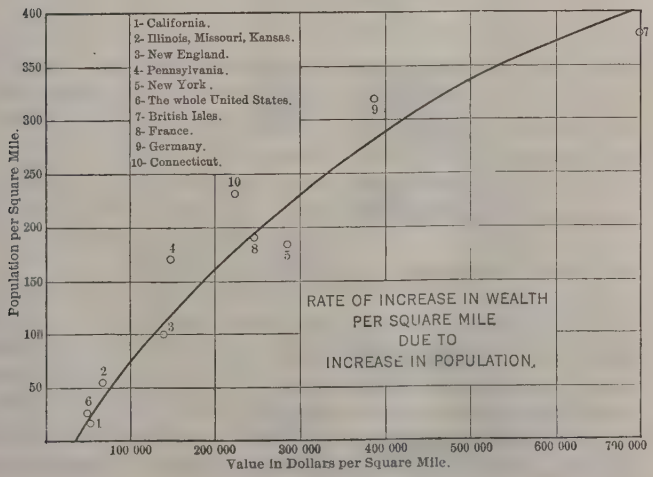
1- California.
2- Illinois, Missouri, Kansas.
3- New England.
4- Pennsylvania.
5- New York.
6- The whole United States.
7- British Isles.
8- France.
9- Germany.
10- Connecticut.
RATE OF INCREASE IN WEALTH
PER SQUARE MILE
DUE TO
INCREASE IN POPULATION.
Population per Square Mile.
400
350
300
250
200
150
100
50
100 000
200 000
300 000
400 000
500 000
600 000
700 000
Value in Dollars per Square Mile.

FIG. 2.

TABLE 1.—Comparative Statistics for the State of California.

	Population.				Valuation.				
	Area, in square miles.	Total population.	Per square mile.	California. Percentage.	Total, based on assessed valuation.	Per square mile.		Per capita.	
						Total.	California. Percentage.	Total.	California. Percentage.
California (excluding from the area desert lands south of Owens Lake)	124 250	2 377 549	19	100	$6 622 000 000	$53 200	100	$2 790	100
Illinois, Missouri, Kansas	208 243	10 622 875	51	37	14 398 850 000	68 900	78	1 350	206
New England	66 424	6 552 681	99	19	9 345 475 000	140 000	38	1 423	196
Pennsylvania	45 126	7 655 111	170	11.2	6 685 500 000	148 000	36	872	319
New York	49 204	9 113 279	185	10	14 035 300 000	286 000	19	1 540	181
The whole United States	3 617 673	91 972 266	25	75	187 739 000 000	51 900	104	2 040	136
The British Isles	121 331	46 064 738	380	5	85 000 000 000	700 000	7.5	1 840	151
France	207 054	39 601 509	191	10	50 000 000 000	241 500	22	1 260	220
Germany	208 780	66 715 000	320	6	80 000 000 000	383 000	14	1 200	232
Japan	147 655	52 985 423	359	5	.....	.....	.....	.....	.....

TABLE 2.—Relative Value of Publicly and Privately Owned Utilities in California in 1912.

Total population of incorporated cities in California = 2 340 242.

Publicly Owned Utilities.

Class of utilities.	Number of plants.	Population of cities reported.	Value of plants in cities reported.	Population of cities owning plants with no value reported.	Total population.	Estimated total value of utilities.
Water	81	1 033 410	$58 633 904	85 986	1 119 346	$67 770 000
Electric	14	109 819	1 884 709	3 700	113 519	1 950 000
Gas	1	6 000			6 000	42 000
Wharves	6	978 127	4 091 440		978 127	4 091 440
Total						$73 853 440

Privately Owned Utilities.

Class of utilities.	Number of plants.	Value.	Total values.
Street railways	30	$454 892 593	Publicly owned $73 853 440
Water	238	204 000 917	Privately owned 1 445 319 496
Gas and electric companies	113	525 677 587	
Telephone and telegraph	81	253 357 061	Ratio of public to private 5.1 per cent.
Wharfinger warehouse companies	87	7 391 338	
Total		$1 445 319 496	

cover this latter item, $26 000 000 is added here. The values for the cities omitted are interpolated by computing the average value per capita for all the cities given and applying this figure to the population of the cities omitted. The Controller's report makes no mention of the San Francisco City Railways. These results are tabulated. Excluding the steam railroads, the value of municipally owned utilities is only 5.1% of those privately owned.

There is, of late, in the United States, a decided trend toward public ownership of public utilities, especially by municipalities. In California, both the Railroad Commission and the Courts have recently prevented privately owned public utilities from entering a municipality which is already being served adequately by another privately owned public utility of the same class. Yet municipalities of this State are exempt from such legal restrictions, and our highest Federal Courts have confirmed the right of the public to invade the field of a privately

owned utility in competition, without the legal obligation either to condemn or purchase the privately owned plants. The people have this inherent right to serve themselves, and though the public should be able to enjoy the profit and security of their own service, there is a broad equity involved that requires the protection of the privately owned plant which has been built in good faith and is being operated under public regulation so that its rates are just and its service adequate. It would be a public misfortune to confirm the impression that private investments are to be over-ridden unfairly and in such a way as to injure legitimate investment. If unfairness is practiced in our larger centers of population, having substantial bonding resources, it is apt to delay greatly the development of the back country, where public credit for large enterprise is lacking.

Rate-fixing commissions must be given credit for stabilizing the value of public utilities bonds. Stockholders have suffered severely, and in many instances they should. In California public utility corporation bonds, approved by State authority, are now sought after. The bondholders are usually only loaners of money. It requires stock buyers to build extensions and new works. The writer feels that it would lead to the more rapid development of the State if the Commission's policy were more liberal in the fixing of valuations, and especially in the permissible rate of interest return allowed. Private capital invested in utilities should be protected against unjust competition of publicly owned utilities. The building of private utilities in California has been practically stopped until it becomes more apparent what this public policy is to be. In Wisconsin this stage of State development has been passed.

In so far as known, neither the California Railroad Commission, nor the California Courts have ever expressed themselves as to what would be a reasonable rate for a publicly owned utility to charge its consumers. The Wisconsin Railroad Commission has reviewed this question, and the trend of its decisions is to the effect that the rate should be fixed in the same manner as though the utility were privately owned, the theory being that, where a publicly owned utility is in competition with one privately owned, it should not be given such preference as to result in the destruction of the value of the private plant. For instance, if a city goes into competition with privately owned plants for the sale of electric energy within its boundaries, and

the interest on the bonds and the sinking fund is paid from the general tax, it would put such a handicap on the privately owned plant as would ultimately work its destruction.

The rates for municipally owned utilities in California have often been fixed rather as a political, than as an economic, question, it being a popular and easy road to public favor to advocate the reduction of any rate.

The rate for a public utility service should be based on the fair value of the properties used and useful in the service, irrespective of whether the plant is privately or municipally owned. The municipal plant should be viewed as an investment of public funds by the city, and it should be operated with the view of obtaining a profit on the investment.

The rate for a privately owned utility should be adequate to provide, first, for the expenses of operation and maintenance, second, for depreciation, and third, for an interest return on the fair value of the property. This last item will provide interest on the bonded debt and any profit accruing to the owner. With a municipally owned utility, in addition to these expenses, there is a bond redemption fund. It has no counterpart in a privately owned plant, as its bonds are seldom retired, but, on maturity, are taken care of by a refunding process. In California, however, the law requires that municipalities retire one-fortieth of the bond issue each year.

There are four classes of people who should assist in bearing the cost of a municipally owned public utility:

First.—The city as a whole, or the government;
Second.—The consumer;
Third.—The owner of vacant lots;
Fourth.—The real estate promoter who desires extensions made to new subdivisions.

First.—The City's Share of the Expense.—The municipality should contribute from the general tax fund for the cost of all public uses of water, notwithstanding the fact that the origin of the fund is usually from the general tax budget. The city should also provide from the general tax budget the annual contributions to the bond redemption fund for the utility. If the municipality's credit has been used in the issuance of bonds for the purchase or construction of the utility

plant, then the refunding of these bonds from the general tax budget is virtually payment for the plant by the city as a whole.

Second.—The Consumer's Share of the Expense.—The consumer should, manifestly, pay for the operation and maintenance of the system, and, if the municipally owned utility is to be treated on the same plane with one privately owned, then it follows that the depreciation account and interest on the fair value of the property should be also charged to the consumer. To deduct from the rates the depreciation and interest would put such a handicap on competing privately owned utilities that they probably could not live. This situation, however, is one that cannot be treated by a hard and fast rule. For instance, if a municipality is looking broadly into the future for its public necessities, as is being done by the Cities of Santa Barbara and Los Angeles in the building of their aqueducts, and the depreciation and interest charges on these large expenditures are put solely on the present consumers, it may result in a rate that would prevent adequate and desired uses of the utility. Apparently, some latitude must be exercised in every instance, in determining how this distribution should be made, in order to obtain a rate which is a feasible operating one.

Assuming that the city can borrow money on its bonds at 5%, and that a reasonable service rate for the utility will provide interest on the fair value of the property at 7%, there accrues a 2% differential which is in effect the profit in operating the utility. This profit should be expended in the ordinary extensions and betterments of the system, so as to avoid the necessity for frequent bond issues to make these improvements. These, in fairness, should be paid for by appropriations by the city council from the general tax fund. However, those who have had to do with these matters know that such allotments of money cannot be obtained, as a practical working proposition. Only when a large and substantial improvement is to be made should it be done by means of a bond issue. If any surplus accrues from the profit of the system, over and above the expenditures for improvements and betterments, this surplus should be turned into the general funds of the city, for the purpose of reducing taxes rather than rates.

There are two classes of consumers under the municipally owned plant, those who are not taxpayers in the community, and those who are. The consumer who does not contribute taxes, is paying a fair price for the service received, and is not entitled to have the cost of

this service reduced simply because he happens to be temporarily a resident in a community owning the utility. He has no investment there, and has assumed no liabilities on its account.

Property owners have guaranteed the payment of public debts. If the bond redemption fund is provided for from the general tax levy, all taxpayers are proportionate owners of the utility. A share of the profits accruing from this ownership should revert to the taxpayers through the general fund, so that the consumer who is a taxpayer receives an indirect benefit from the ownership of the utility in this way. This system would encourage property ownership.

Third.—The Vacant Lot Owner's Share of the Expense.—The vacant lot owner in our Western towns is a source of great expense and burden to all utilities. It is probable that one-half the mileage of distribution mains is caused by his speculation in land values. The utility is standing ready to serve at any time, and the value of the vacant property depends largely on its privilege to be benefited thereby. Unless a charge is assessed against vacant lot owners, they are securing benefits without adequate contributions to the expense of the plants. Some system of assessment on such vacant property on a front-foot basis should be developed to cover this element, as is often done in street paving and sewer work.

A portion of the cost of building the utility is borne by the vacant lot owner in that he is a taxpayer and has assumed the liability for the debts of the community. He enjoys the broad and general benefits that result from the operation and ownership of the plant, which the consumers alone should not pay for. There are many vacant lots in the old and established portions of the city where the plant has long been built, and such lot owners cannot be assessed as in the case of an extension to a new subdivision. Possibly they should be charged an assessment each year, which should go into a fund for ordinary betterments of the system or into the public treasury.

Fourth.—The Real Estate Promoter's Share of the Expense.—The real estate promoter is continually making new subdivisions and petitioning the utilities, whether publicly or privately owned, to make extensions therefor. With privately owned utilities it is usually demanded that the promoter shall pay for the cost of the extensions. This policy has also been followed in many of the publicly owned utilities, but it has resulted in political agitation, and has been vigorously

resisted by the interests who are active in their efforts to speculate on the community.

The vacant lands exploited by the real estate promoter may be divided into two classes: first, those lying within the corporate limits of the municipality owning the public utility; and second, those outside these corporate limits.

In the first instance it would be equitable to require the owner of the property to pay a frontage tax to cover the cost of the distribution system fronting the property in question, this contribution to be returned when this part of the plant is on an earning basis. It would not be proper to charge a portion of the cost of the mother plant against these extensions, because the property will already have paid some proportion of the cost thereof in taxes, and will pay more as its value increases. The assessments for the extensions to the new subdivision within the corporate limits of the city should be in the nature of a temporary donation.

If the lands to be exploited are outside the corporate limits of the community, it would again be just to require the owner temporarily to advance the funds necessary to make the extensions. So far as concerns storage reservoirs and conduit systems, or any of the mother plant built for the distant future, and which may equitably be assumed to involve the promoter's lands, a portion of the cost of this excess capacity should justly be charged to the promoter.

This whole problem cannot be solved rigidly with a rule which will apply to all cases. There is, however, the necessity for outlining some general theory, on which municipalities should proceed. Even if it is clearly defined, and is endorsed by high authority, there will be the practical political difficulties of fixing and maintaining rates at a just point. Obstacles will have to be overcome in case it is necessary to increase a rate. There is great inertia inherent in communities, and this it is necessary to prevail against, in order to make a change of any sort.

Our cities are becoming great business concerns, and should be managed fairly and scientifically, not only for the interests of the citizens therein, but also for those who have financed in one way or another their utilities and early development.

DISCUSSION

Mr. Harris.

FRANK S. M. HARRIS,* ASSOC. M. AM. SOC. C. E. (by letter).—Mr. Lippincott has taken a thoroughly commendable stand in insisting that municipalities enter a public utility field which is more or less adequately served by a privately owned utility under rate conditions equivalent to those confronting the private enterprise. It is only as a result of such an equitable attitude that private capital may be expected to "pioneer" in those fields into which the public is not for the time desirous of entering. It is decisions such as that in the hearing of the City and County of San Francisco in the matter of special crossing privileges for its Municipal Railways,† in which the municipality was reduced to the standing of a private corporation, which have operated to remove the bitterest of the criticism heaped on the heads of utility commissions by the public service corporations which have been "regulated".

The writer has but recently completed an exhaustive survey of the whole franchise situation for the City of Oakland, Cal., in the course of which there was indirectly revealed the devious if not perilous path traversed by the average public utility company prior to 1900. The manifold injustices, at the hand of shifting political régimes under which they labored, were but equalled by the corrupt advantage which they in turn took of the municipality. It is refreshing, therefore, to find set forth so fair a basis for square dealing between the municipality and the private corporation.

Mr. Lippincott, however, considers only the case in which the municipally owned utility exists side by side with, but completely independent of, the privately owned utility.

Of late years, an innovation is being written into the franchises of privately owned utilities, in which the municipality is an active partner in all development and extension programmes, as well as in the formulation of financial and operating policies. Such a provision is but a corollary of the principles advocated by Mr. Lippincott.

For the new blanket resettlement franchise, now being sought by the San Francisco-Oakland Terminal Railways in exchange for its 130 or more parcel franchises, the following provisions are found:

(1)—A board of control composed of one representative of the city and one representative of the company shall have jurisdiction over such matters as extension, outlay for equipment, allowance for depreciation, and the like; and

(2)—After payment of operating expenses, taxes, depreciation as fixed by the board of control, and a 6% return on its agreed

† Oakland, Cal.

‡ *Engineering News*, Vol. 74, p. 181.

Mr. Harris.

physical valuation, at least 55% of the net revenue remaining shall be paid to the city.

These clauses are in addition to the usual machinery of the indeterminate franchise, providing for the taking over of the ownership by the city on 6 months' notice at the agreed physical value.

It would seem that, until such time as public ownership is the accomplished fact which it now bids fair rapidly to become, either an equitable rate-making policy for the municipal competitor or an implied co-operation between the municipality and the privately owned public utility are the essentials of any thoroughly satisfactory solution of the existing difficulties.

Mr. Thomson.

T. KENNARD THOMSON,* M. AM. SOC. C. E.—Mr. Lippincott deserves the thanks of the Society for his very clear presentation of a most vital issue.

We are going through a formative stage at present with such subjects as socialism, suffragettism, unionism, public ownershipism, anarchism, and general radicalism, and the tendency of the country is to try everything, even including Wall Street speculation; but, fortunately—like an intelligent child who has burnt his fingers—the country profits from the expensive experience, and tries something else. For instance, a town will elect a Socialist mayor, and then snow him under at the next election. Again, a State will try the "referendum and recall", pass a dozen different and contradictory bills on the same day, and then try to recall both the "recall and the referendum". The experiments which prove profitable will remain, and even a few, like suffrage, will undoubtedly stick, if adopted, whether really beneficial or not.

When it comes to public ownership, we have examples in New York City—the Staten Island Ferry, Brooklyn Bridge, etc.—which, as everybody knows, are not financially profitable. Even the Rapid Transit would not pay if entirely owned by the municipality. For what city would dare to pay a yearly salary of $100 000 or more to one man, even if he could earn many times that much for the taxpayers.

Ninety-nine men out of a hundred will say that no man is worth more than, say, $10 000 a year, and that he (that is, the other man) has dozens of men under him who could do the work as well or better for less money; and this in face of the facts found actually all over the world, for it is hard to make people believe what they see.

To take a single example: The Intercolonial Railroad, one of the best built railroads in America when constructed by the Canadian Government, enjoyed an absolute monopoly between Halifax and Quebec for many years. True, the country was largely undeveloped,

* New York City.

Mr. Thomson.

but not as much so as the vast Canadian Northwest was when the late Sir William Van Horne finished the Canadian Pacific Railroad; but Van Horne saw to it that the great prairies were developed to build up his road; whereas the men who ran the Government-owned Intercolonial stuck strictly to the business of operating the road, instead of creating business to make the road pay, with the result that for some 40 years the road did not even earn operating expenses, let alone pay taxes or interest on the money.

The speaker, when recently talking to a prominent Canadian, suggested that, if the Government had engaged a man like the late Sir William Van Horne, or the late James J. Hill, F. Am. Soc. C. E., or any one else of that caliber, and offered such a man a salary of, say, $50 000 plus a bonus, he would have earned his salary, a bonus of several hundred thousands a year, after having paid the Government ample taxes and a good interest on the money invested in the railroad, and, at the same time, would have developed the adjoining territory, thus benefiting the country from one end of the Dominion to the other.

My friend contradicted me, saying that the Government at one time really did engage a man capable of getting these results, and that he at once began by examining the freight rates, which disclosed the fact that many of them were ridiculously low. He ordered the rates raised and made the same for all. To make a long story short, his days were numbered, and the road ran on as before.

On repeating this story to a very powerful newspaper owner, the speaker was told that such a state of affairs was quite justifiable, because it paid to run a road at a loss for the sake of building up the country. This, however, is exactly what such management never does, as it considers its duty done when it operates the road.

This is no reflection on the staff of this railroad, which is a very competent one, whose work is well done—that is, as far as it has any authority to work—but one man with the necessary power, and inducement, could take the same staff and turn a financial failure into a great financial success, thereby benefiting the entire country.

To take another example, nearer home; a fatal mistake was committed in the construction of our magnificent $150 000 000 Barge Canal (with political "poorsight") in the beginning when it was made a barge canal, instead of a ship canal. As it is, no boat with a clearance of more than 15 ft. above the water can pass under the bridges and use the canal. This mistake was not made on the Canadian canals, which, with the same or less depth of water as the Barge Canal, are able to pass vast fleets of masted vessels.

It now remains to be seen whether the same lack of judgment will be displayed in the operation of the Barge Canal, as on this depends the profit or loss on the enormous sums paid by the people for this undertaking. If the canal is properly managed, the State will enjoy

Mr. Thomson.

far-reaching results. Moreover, if a man is employed who has the ability and is given the power to make business for the canal, as well as to see that it is properly operated, the State will be handsomely rewarded for the sums expended for the construction and maintenance of this work, which in many ways far surpasses the Panama Canal.

It is just as if the State were to build a number of huge skyscrapers and then trust to luck that the people would find, use, and pay for them. Needless to say, the privately owned buildings adjoining, run by wide-awake business men, who would go after their tenants, would be filled at once, while the State buildings would be nearly empty, and of course operated at a loss.

It seems to the speaker that by discussing actual cases more progress will be made than by the expression of purely theoretical opinions, and that the ultimate result will be proper Government regulation, instead of Government operation or ownership.

Last spring, at a political meeting, called for the purpose of explaining the extra $19 000 000 appropriation in New York State, the speaker, after the regular speakers had finished, begged to draw attention to the great difference between the City and State on the one hand, and an ordinary business organization on the other; the business man has to earn the money he spends, while the City and State spend the money which they force the tax-payers to contribute.

The speaker then pointed out two projects which, if encouraged by the City and State, would result in putting both on a business basis. The City and State would receive more than $30 000 000 outright and a rental of at least $2 000 000 a year, to say nothing of enormous indirect benefits. The projects are "A Really Greater New York", and a "New Niagara Falls, of 2 000 000 h.p."

Mr. Yereance.

W. B. YEREANCE,* M. AM. SOC. C. E.—The author has stated clearly some of the problems to be met in the municipal ownership of public utilities. It is assumed that he has contemplated as well municipal operation of those properties or plants. Of course, it may be, and so happens, that some properties are municipally owned but privately operated by private corporations.

An insolvent or mismanaged utility is a handicap to the progress of the community or district it serves. A utility should be in receipt of a sufficient revenue to meet, under proper management, all operating expenses (including maintenance, depreciation, taxes, and a sufficient reserve as insurance against catastrophe), and a reasonable return on the fair capitalization of an adequate plant. This revenue is or should be produced practically in its entirety by the rates charged for service. Rates are or should be determined so that each beneficiary contributes to the total necessary revenue of the utility only his just

* New York City.

and due proportion for benefits received. If the plant be ill-adapted to the community's needs, or unnecessarily expensive, or the management extravagant or incompetent, or the securities inflated, the rates to produce the needed revenue will be unnecessarily high or unreasonable, either throughout or for certain classes of the service. Therefore, if minimum rates are to be enjoyed, the property or plant furnishing the service must be designed and constructed wisely and economically and operated efficiently. To the extent that these conditions have not been met, the rates will be above a possible minimum. Mr. Yereance.

For a given standard of plant and service in a given community, the rates for that service should be lower under municipal than under private ownership and operation. Yet, despite some highly flattering operating statements issued periodically, the speaker knows of no situation on this continent to-day where municipally operated utility service is being rendered at actually lower rates than are or should be charged for similar service by private enterprise. If politics and political considerations can be eliminated in the development and operation of a public utility, then and then only will be possible the successful establishment of actual rates that to-day are merely a dream of the theorist. Evidence is not lacking that there are those who realize the situation, and it may be that at some future period the ideal will be realized. However, those now interested in the private administration of public utilities need have no fear that their retainers or jobs are in immediate danger. If that ideal time comes when the public utilities can be removed from adverse political influences, then public utilities should be municipally owned and operated. Private enterprise has its own problems to meet in the conduct of public utilities, but it could not live under many of the practices now associated with municipal ownership and operation.

The author has referred to the inherent right of a community to serve itself, but calls attention to the broad question of equity involved where such contemplated municipally owned and operated service would introduce competition with a private plant, established in good faith, and serving the public adequately at reasonable rates. Some regulatory bodies have recognized the ethics of such a situation in their decisions, and the demands of simple justice—to say nothing of policy—have been met; other commissions have not been so far-sighted.

Some 3 or 4 years ago a campaign favoring municipal ownership was waged vigorously by some of the ambitious politicians in one of the Southern States. At that time the speaker was retained to protect a considerable investment of northern money in a combined electric light and gas property in one of the cities of that State. The electric plant was probably the most up-to-date of any in a community of that or considerably larger size in the United States; the gas

Mr. Yereance.

plant was not in quite such good shape, but both were fully adequate, and the rates charged were lower than prevailed—under Commission authority and at some places under more favorable conditions—in any community of similar size in that or adjoining States. The good faith of the investors was apparent in both the condition of the property and the amount of money actually invested, yet the political elements favoring municipal ownership practically forced the company into a desperate position, whence it was extricated with the greatest difficulty.

The attitude of the commission, the public press, and many of the people, was such as to discourage the investment of outside money in that locality; some are now beginning to realize the short-sightedness of that attitude when new money is desired but not procurable. Certainly, in the light of these facts, no honest consultant could or would recommend investment where such conditions prevail.

Reference has been made to a certain steam railroad, governmentally owned and operated. Located through thin traffic country, its value would seem to be largely military rather than economic. Such property needs a higher grade of management than does one which is more favored; yet the road in question has always been notoriously politically mismanaged. It could have been operated better with one-half the number of men shown on its pay-rolls. Instead of the two or three—as mentioned by Mr. Thomson—there were at least ten times that number of good railroad men in the United States who, with free hands, could have developed the full earning capacity of the property.

Mr. Jordan.

Leonard C. Jordan,* Assoc. M. Am. Soc. C. E. (by letter).—A perusal of this paper conveys the impression that the author is in favor of private ownership of public utilities, and that he is even willing to go beyond the bounds of justice in hampering municipal ownership. The writer agrees with the author in advocating private ownership of public utilities wherever such a course is advisable, but he begs leave to differ with him in one or two of the points to be mentioned herein.

The author has brought up the matter of the status of the owner of vacant lots and his responsibility to the community, so far as respects his advantages from public utilities. It is quite true that he has distinct advantages in the increased value of his holdings, and it is equally true that he has been an important factor in running up the cost of these utilities by causing them to extend their service lines past his unused property, in order to reach the used property of other owners, the people who, by making use of the utility, furnish the revenue which keeps it in existence and on a profitable basis. However, there is probably greater likelihood of general all-around

* Morsemere, N. J.

justice in this respect when the utility is owned by the municipality, for, in the case of the municipally owned utility, the expense is met largely by assessment against the abutting property or by taxes on property in general. In the case of a privately owned street railway, for instance, the owner of the vacant lots very probably would fall heir to all the latent benefits of the railway without the expenditure of a single penny. Furthermore, his neighbors must pay, by the revenues from their fares, his share of the expense of construction and maintenance, as well as for the unwelcome privilege of being hauled past his unsightly and weed-covered property. Although the author advocates private ownership, this problem which he introduces would seem to be regulated more justly for all concerned when every utility is publicly owned and properly managed. Mr. Jordan.

There is a contradiction of principle cropping out at various places in the paper, which is summed up very concisely in the Synopsis, as follows:

"If the rates for service of publicly owned utilities are too low, and deficits are made up from the general tax funds, such administration is unfair to privately owned utilities, and will tend to discourage the investment of private funds therein, in States practicing this policy."

Then a few lines farther along:

"Any surplus resulting from such administration [municipal ownership] should be deposited in the public treasury of the city to the credit of the general tax fund."

If it is proper to turn a surplus into the city treasury, then any deficit from operation should be remedied by drawing on the general city funds. Probably a correct business system of managing a municipally owned utility would include the keeping of strict accounts of all money matters pertaining thereto, as though it were an industry in itself and distinct from all other parts of the city government. The city treasury should be regarded as a bank, lending money to tide over unproductive periods, accepting surplus funds on deposit, and handling the bonds when their issue is required by unusual construction or development expenses.

Again, on pages 420 and 421 the author advises the charging of rates higher than the nature and business of the utility would justify and the turning of surplus amounts into the city treasury. It is argued that this method is fair to the taxpayer as it reduces his taxes on real estate. The writer is unable to see any justice in placing a burdensome portion of the expense on the poorer people of the community, in order that the property owners may enjoy the advantage of reduced tax rates. Such procedure would tend to break down democracy, in that it places obstacles in the way of the poor man and makes

Mr. Jordan.

easy the way of the man who already is rich. It is stated, relative to this matter, that the non-taxpayer is paying a fair rate for the services received, even though he is paying for a portion of the taxes of his more fortunate neighbor, but the writer fails to detect much fairness in the plan. We might as well have municipal ownership of grocery stores, charge the high rates that would be possible under such non-competitive business, and then attempt to square matters by cutting down the taxes of those people who happen to own their homes. It is far more just, for the individual and for the community in general, to let each man pay his own way as he goes, than to attempt to equalize high rates for trolley rides and for electric lights by reducing the taxes on the same vacant lots which cause so much extra expense to utilities by increasing the length of their service lines.

The writer believes that a fair and strict accounting of the construction and operating costs of publicly owned utilities would furnish sufficient argument to convince the most stubborn that private ownership is the best plan under present conditions. He would go further in the accounting, however, than is generally done. When summing up the construction costs, all contributing factors should be included, just as they would have to be if the work were being carried on by a private company. This would require that salaries, office rent, and other expenses of public officials connected with the work be included in the bill. It would require also that depreciation of equipment and interest on its investment be reckoned, and that interest on the cost of plant and on invested construction expense be computed and included.

In compiling the accounts of the cost of operation, likewise, every part of the cost should be hunted down and charged in the proper manner. If the city has a "Commissioner of Public Works", who manages the municipally owned water-works system, then his salary and a fairly computed office rent for his rooms in the City Hall should be charged against the operation of the water-works. Furthermore, if a quarter of the time of the city treasurer and his staff is required for collecting and handling the water rents, then a quarter of their entire expense should be charged against the water revenues; and, finally, the usual tax rate should be levied against the water-works, and the tax should be paid into the city treasury and charged against the earnings of the water department. This matter of taxing might seem to be superfluous, at first thought, but a moment's consideration should suffice to convince the reader that it merely means a transfer of certain figures from one account to another, adds a negligible amount to the cost of city government, and aids in showing exactly the financial condition of the water-works department. Such a plan of accounting would form a correct basis for the regulation of rates, and would be entirely fair to the private owners of rival systems.

Besides, the strict business system would furnish an incentive to efficient service on the part of the men employed in the department. Mr. Jordan.

As a further example, a battleship is constructed at a Government Navy Yard for a certain figure, while a shipbuilding company builds the sister ship on contract at a greater stated cost. The newspaper comments mislead some people into the opinion that the only lack of perfection in the Navy is its failure to construct both boats. Yet, if all costs were tallied, the total might result in a far different opinion. The figures for the Government-built boat should include:

All the costs of plans, including the salaries of officers who oversee the work of preparation;

Office rent for all men engaged on this work, even though they occupy rooms in a Government building;

All construction costs that would be encountered by a private company doing the same work;

Salary, keep, barracks rent, etc., of all marines doing police duty in and about the ship until she is put into commission, and this item should include the expenses of officers in command of these marines;

Reasonable rent for that portion of the navy yard that is devoted to the job, or else interest, depreciation, maintenance, and taxes on the portion used;

Interest on the investment, depreciation, and maintenence of the equipment used;

Salaries, office rent, and other expenses of all officers and men connected directly or indirectly with the work;

A portion of similar expenses for all men whose duties are divided between this and other jobs, even including the Secretary of the Navy, if a portion of his work may be regarded as arising from this source;

Interest on all the foregoing amounts, computed to the day that the boat is finally put into commission;

And all other items, great or small, which directly or indirectly contribute to the amount which the Nation must finally pay for the battleship.

Some of these items are not included in the stated cost of the boat built by contract, but they should be added. The justice of a complete cost-keeping system will be realized when the reader considers the possibility of the purchase of a finished battleship from a foreign country or firm, as several other countries have done within the past few years. Besides, the people should have a right to know the total amount which such a fighting machine is costing them.

It might be contended that the office rent of the Secretary of the Navy would seem unduly high if computed on the basis of the cost of

Mr. Jordan. the building in which it is located and the cost of operating that building. If that is true, then the people have a right to know that they need a new janitor in that building, or a new building superintendent, or else that they paid too much for the building originally. From another angle, if that office rent is higher than would be charged for suitable quarters elsewhere, then there is first-hand evidence that the department occupying these offices is not qualified to be placed in charge of a business requiring a high grade of efficient executive ability, such as a shipbuilding establishment. It remains that the people have paid for the building and are continuing to pay for its upkeep, and it would look like poor management on the part of the Government if there is no possible return on the first cost and running expense. Besides, failure to account for all such matters would be unfair to the contracting firm which is willing and anxious to put together a battleship for less money than it really is costing the Government.

The result of such complete cost-keeping methods as suggested would probably lead to private ownership of public utilities, such as the author seems to prefer, a private company having a monopoly of the patronage of the community and correctly regulated by just officials of the local government. This is the ideal form of service when both private owners and ruling officials are ideal; and let us trust that we may enjoy these conditions at some time in the near future. At least, let us hope that any owning or regulating on the part of the Government will be done better than it seems to be doing with any such work now. Meanwhile, with all fairness to all parties concerned, let those engineers who are in position to decide or to influence questions of municipal or private ownership of public utilities render their decisions with impartial judgment, having in mind the furtherance of principles of justice which too often are overlooked in such affairs.

Mr. Clark. H. F. Clark,* Assoc. M. Am. Soc. C. E. (by letter).—The author has presented views on a subject which engineers are now having to consider seriously. Where public utilities are owned by cities, it is generally found that the cities prefer to have their own way in determining how the costs of operation shall be spread, and rather resent the intrusion of scientific suggestions. The tendency is to better management of civic affairs, with centralization of power in one man or a small group of men. Engineers may yet be of some assistance to the budget makers of a municipality in determining the questions which the author has brought out.

Municipal ownership in California is steadily spreading, records for 1916 showing approximately fourteen transfers of water systems to municipalities. The aggressive element within a community generally lays siege to the water company until at last bonds are voted

* San Francisco, Cal.

and the water system is purchased. A feeling of security then comes over the minds of the people, as no longer will there be occasion for petty annoyances. The constant friction over service and rates is then passed, and, be the results profit or loss, there is always that true friend, the tax budget, to come to the rescue in times of need. A monopoly exists over the community when this competition is removed, and the city finds itself a sovereign in the matter of the regulation of this utility business. Mr. Clark.

Analysis of the paper involves accounting to a large extent. All are aware that the full cost must be met somewhere, it remaining to be determined how it shall be distributed. To bring out a little more clearly the distinctions which the author has made, Table 3 shows his distribution of the expenses.

TABLE 3.—Distribution of Water-Works Cost in a Municipally Owned Utility.

Item of expense.	Class Bearing Expense.			
	Municipality.	Consumers.	Vacant lot owner.	Realty promoters.
Maintenance and operation	Through taxes for public uses.	Through rates.	.....	.....
Depreciation	.....	Through rates.	.....	.....
Interest	.....	Through rates.	.....	.....
Extensions	.....	.....	.....	Through deposits.
Betterments	.....	.....	Through special tax.	.....
Bond redemption.	Through taxes.	.....	.....	.....

The author states that municipalities have an item of expense to bear, in addition to those expenses borne by a privately owned utility, namely, the redemption of bonds within a 40-year period. The author suggests that this redemption fund should be met in the general tax levy, and therein, the writer thinks, he is in error. Cities, admittedly, are enabled, through their credit, to borrow money about 2% cheaper than a private company can finance its operations. If the city makes a 7% interest charge, as would be granted to a privately owned utility, any profit resulting under the bond rate of interest could be laid aside to redeem the bonds. If the bond rate of interest is 5%, the differential of 2% compounded annually at 4% will approximately retire the bonds in 30 years. Thus, as a fact, there is apparently no additional expense, but, in the case of a municipally owned utility, merely one more division of the same gross expenses that would be allowed a private corporation in the same utility business. The author has suggested the possibility of a profit in the management of the utility. Occasionally, reports made from an audit of the books show profits,

Mr. Clark.

but it is rather the exception to conditions. The writer does not see why there should be any profits under the support of municipal ownership, if the rates are adjusted scientifically. It should not be the aim of a city to make its water consumers pay more than the cost of the service. Too often the consumers cannot even bear this cost, and part of it is carried under other accounts into the tax rate for the year.

Table 3 places on the consumers the depreciation charges. There is no more depreciation accruing annually in front of an improved residence lot than in front of adjoining vacant property, eliminating the item of service connections and meters. It is not clear to the writer why only the users of water should be burdened with the depreciation account, when the vacant property is increased in value, due to the presence of the main in the street. Municipal ownership is generally favored when the condition of the city as a whole will be benefited, rather than the individual condition of each consumer. Though it is true that, under a privately owned utility, the consumers have to bear this depreciation, yet it is fair, under municipal ownership, that the vacant property should bear equally with improved property this item of expense.

The distribution of the various items of cost would appear to the writer to be more equitable if as shown in Table 4.

TABLE 4.

Item of expense.	Class Bearing Expense.			
	Municipality.	Consumers.	Vacant lot owners.	Realty promoters.
Maintenance and operation	Through taxes for public use.	Through rates.	.....	.....
Depreciation	Through taxes.	.....	.....	.....
Bond interest	.....	Through rates.	.....	.....
Bond redemption	Through taxes.	.....	.....	.....
Extensions	.....	.....	.....	Through deposits.
Betterments	Through bonds or revolving fund.	.....	.....	.....

In California it has generally been found that the maintenance and operation expenses of a water utility will approxmiate 40% of the total expenses chargeable. By the foregoing classification, an analysis will show that the consumer divides with others about evenly with the remaining 60% of the expenses which occur.

However, the writer agrees with the author that no rigid rules can be formulated in this matter, inasmuch as there are so many local situations to be met, as is shown by occasional items in the newspapers. Not long ago, an article commented on the refusal of a water-works board to turn over a considerable sum of money to the general city treasury,

in order that the tax rate might be decreased. The reasons given for the refusal were that the money would all be needed for improvements, in view of a contemplated reduction in the minimum monthly charge for water, with correspondingly less revenue for the year. This minimum was already one of the lowest in the State, but yet it seemed advisable to that board to lower it still further. Again, in another city, lately adorned to attract visitors, the park department began to use enormously increased quantities of water, drawing heavily on the safe yield of the system. The water superintendent was at his wits' end to keep up with the demand, and was unable to make any showing for his department, on account of the extreme demand of the park department, which gave him no credit for the water used. Mr. Clark.

The abuses of the sovereign water plant management may be corrected in time by the application of principles such as the author has presented. There is great need for city officials to recognize just such principles.

Allen Hazen,* M. Am. Soc. C. E. (by letter).—This paper is timely and important. It represents a swing back of the pendulum that was set in motion when it was decided that henceforth the work of corporations in certain lines should be considered as public and not private business, and that it should be subject to public control. Mr. Hazen.

The control of such corporations by public bodies is directed primarily and principally to the control of rates. The work of reducing rates in the interests of those served, formerly carried out occasionally by legislation or by Court action, is transferred to commissions created for that special purpose. The business is ordinarily expedited. It is handled by men who at least have the experience growing out of handling a considerable number of such matters. Regulation of the character of service also is undertaken, and though by no means as complete, something is accomplished.

Quite recently, perhaps since Mr. Lippincott's paper was written, another and far-reaching step has been taken; for, in one important case, Government regulation of wages paid to workers employed by such corporations is undertaken.

There is one matter, however, which cannot be regulated by Government; investors cannot be made to put more money into an enterprise, because better service is needed, or because existing rates do not furnish the money necessary, or because money is needed to pay increased wages.

When the reduction of rates and the imposition of other burdens on public service corporations has proceeded to a point where investors will no longer furnish money to carry out needed improvements, it has clearly gone too far for the good of the business and the community.

* New York City.

Mr. Hazen.

If conditions of regulation become such that private capital will not be supplied willingly to carry on the business, the only apparent way by which the public can control it further is to take over the undertakings by the right of eminent domain, and raise the needed money by taxation or by borrowing on the public credit.

It is to be noted that, in the course of the procedure which results in reducing a corporation's profits, the last step of profit reduction comes at a greatly accelerated rate, by reason of a condition which may be and probably often has been overlooked. When a corporation is making fair profits, its credit is ordinarily good, and it can borrow money at the lowest market rates. As its earnings are reduced to a point which reduces profit below normal—and, perhaps, threatens to wipe it out—the corporation's credit is impaired, and the rate at which money can be borrowed is increased.

If the profits are good, the first reduction in rates may not have this full effect, but, as reductions proceed toward the point of eliminating profit, the double effect is felt, and the final profit is reduced and eliminated quite as much by the increased cost to the corporation of getting money as by the direct loss in rates.

It is a conservative estimate that, as a result of the severe handling of corporations in some of our States in the last years, the ordinary rates which well-managed corporations have been compelled to pay in order to secure money to carry on this business, has been at least 1% per annum higher than it otherwise would have been.

To illustrate: If a corporation is earning 7% and borrowing money at 5%, it has a surplus of profit of 2%, which is a margin tending to make the business attractive to new investors and to maintain the corporation's credit. The margin is not wide, but it may answer. If in such a case, the gross earnings are 14% and the operation cost and taxes are 7%, and there is a reduction in rates, which effects a reduction in gross earnings from 14 to 13%, the direct effect is to reduce the net return from 7 to 6 per cent. The indirect result—which may not follow quite as rapidly, but is no less sure—is that, with the reduced margin to sustain credit, it is no longer possible to borrow money at 5%; and instead 6% must be paid, all the profit is wiped out, and the corporation is on the way to bankruptcy. A reduction of only 7% in the gross income may mean the difference between a margin of profit which will enable the business to be conducted successfully on conservative lines, and bankruptcy.

In the long run, the question of fair returns is boiled down to this: The returns must be sufficient to allow the business to be conducted successfully and permanently, and to permit everything to be done that needs to be done, including the securing of all new capital necessary for the reasonable extension of the business.

Mr. Irving.

W. G. IRVING,* ESQ. (by letter).—The writer has attempted to give due consideration to the conclusions expressed in this paper, but has no doubt that more study would be profitable. He agrees with the author that a municipally owned utility should be viewed in the same light, and should have the same restrictions and privileges, as a privately owned public utility.

The construction or operation of a public utility, such as waterworks, lighting works, etc., is not a municipal function. A city, as such, has no inherent right to engage in such forms of activity.

It is merely by special dispensation of the sovereign power that municipalities have been permitted to enlarge the scope of their powers to include the ownership and operation of utilities; and, in the operation of these utilities, with the exception of the fixing of rates, the municipality is impressed with the same obligations, as to service and as to responsibility for damages, as is a private owner of a public utility.

Therefore, the writer is of the opinion that the municipally owned public utility, in fairness to the privately owned public utility, should be under the same rate-fixing authority, and that the fixing of the municipal rates should take into consideration the same factors as are considered in fixing the rates of privately owned public utilities.

The writer, however, notes in this paper that the author has defined four classes of people who should contribute to the cost and maintenance of a municipally owned public utility. The writer is inclined to the opinion that this classification is arbitrary. No such classification is made with reference to privately owned public utilities. The city should either authorize a bond issue or make a direct appropriation from its funds, to provide the capital necessary to construct the utility. Then rates should be fixed by the constituted authority, to be charged the consumer for the service to be rendered, which should provide sufficient return to pay interest on the investment, cost of operation and maintenance, and to cover depreciation. There should not be, however, any amount allowed as a sinking fund to retire the bonds, if such are issued. The bonds should be retired by a direct tax on the taxable property of the city. As the bonds are retired, the interest payable thereon is decreased, and a greater proportion of profit of operating the utility should be paid into the general fund of the city which would permit a *pro tanto* reduction of taxes.

By this method, the owner of vacant property will be compelled to pay his portion of the cost of the system, and will also derive benefit therefrom, by reason of the enhanced value of his property and the ultimate reduction of his taxes by the profits earned by the utility.

* Riverside, Cal.

Mr. Irving.

The vacant lot owner, and all others who do not receive service from the utility, will also have to contribute to the maintenance of the system and to the payment of interest on bonds (if such are issued), during the adolescent period of the utility, as during that period the actual consumer cannot be expected to pay more than a fair rate, having in view the ultimate service to be rendered and the present capital expenditure.

By pursuing the foregoing method, a fair division of the general expenses of a publicly owned utility could be made, and, at the same time, it would prevent unfair competition with privately owned public utilities.

In the foregoing the writer has considered the question on general principles only, and has not had, and naturally could not have, in mind the many practical difficulties and scientific factors which would readily enter into the author's consideration of the question.

Mr. Whitney.

H. A. WHITNEY,* M. AM. SOC. C. E. (by letter).—Inasmuch as the underlying principles that go toward setting the service rate for one municipally owned public utility may be applied quite readily to all municipally owned utilities, the writer will confine his remarks to the utility of greatest importance, *viz.*, "the water department".

In order to estimate the annual charges against a water system thus owned, the following items should be known:

Schedule I.

(1)—The reproduction cost of the system, less the depreciation;
(2)—The annual depreciation of the system;
(3)—The annual cost of operation and maintenance;
(4)—The cost of business management (generally included in (3));
(5)—The amount of interest to be paid annually;
(6)—The amount to be set aside for new construction.

These annual charges will be divided into the following classes:

Schedule II.

Item 1.—Charges to the municipality; bond issues for construction;
" 2.—Charges to local improvement districts for local construction;
" 3.—Charges to the municipality to be paid from general funds;
" 4.—Charges to consumers for service installations and extensions;
" 5.—Charges to consumers for domestic rate;
" 6.—Charges to consumers for manufacturing purposes.

* San Francisco, Cal.

Mr. Whitney.

The principal advantages gained by a municipality in owning its utilities are:

Schedule III.

(1)—Lower interest rates than those generally allowed privately owned utilities;
(2)—Lower water rates to consumers, due to vacant property helping to pay for the bond issues, and other charges paid for by the municipality;
(3)—Privilege of having ample fire protection, and thereby getting:
(4)—Lower insurance rates;
(5)—Power of extending the system where it can do the greatest good to the greatest number of people.

The foregoing are generally cited by advocates of a municipally owned utility as benefits derived by the municipality which owns its own system. Although, in a good many cases, a low water rate can be obtained by the consumer at the expense of the entire city (it bonding itself to pay the sinking fund and interest on the cost of the whole system, and the construction being permanent), yet, as in the case of the City of San Diego, the city was quite willing to do this, as was shown in 1912, when, by a vote of 6 to 1, the city purchased the impounding system, and again, in 1914, when, by a vote of more than 5 to 1, it decided to purchase Morena Dam. Furthermore, the heavy taxpayers were all back of the project.

Santa Rosa, Cal., went even further than this, a few years ago, and, by a large majority, voted bonds to purchase and operate a water system—the consumers to have free water.

The total annual revenue and charges should be such that they will equal the last five items in Schedule I. A list of these charges is set forth in Schedule II, and will be taken in their regular order.

Item 1.—Charges to the Municipality; Bond Issues for Construction.—As a main trunk sewer, a main drainage canal in a municipality, or a trunk highway, is a benefit to a community as a whole, so should dams, main pumping plants, and transmission lines be so considered; and the costs should be paid by the municipality, by either a tax levy or a bond issue.

Item 2.—Charges to Local Improvement Districts for all Local Construction Which is not a Benefit to the Community as a Whole.—Under this item all distributing mains, service mains, local reservoirs, and stand-pipes would be assessed against the district benefited.

The following sections from Chapter 98, Sections 3, 5, and 15, of the Local Improvement Laws, State of Washington, for 1911, will illustrate quite aptly the privileges derived by being able to apply these laws. The following are extracts from certain portions of these laws:

Mr. Whitney.

Section 3 allows the city to provide for sewer, drain, and water systems, within or without the city limits.

Section 5 allows the city to provide for protection from fire by an auxiliary water system. Besides allowing the construction of the ordinary distribution system, the following paragraph allows the construction of drains of any size, either within or without the city limits.

"Section 15.—Any city shall have power to provide for the construction of trunk sewers, and trunk water mains, and for the payment of all or any part of the cost and expense thereof by the levying and collecting of assessments upon property specifically benefited thereby. In any such case the district created to bear such assessment shall be outlined in conformity with topographical conditions, and in case of trunk sewers, shall include as near as may be all the territory which can be sewered or drained through such sewer and sub-sewers connected thereto, and in case of water mains, shall include as near as may be all the territory in the zone or district to which water may be distributed from such trunk water main through lateral service and distributing mains and services. In distributing such assessments, there shall be levied against the property lying between the termini of the improvement, back to the middle of the blocks along the marginal lines of the street or area improved, such amounts as would represent the reasonable cost of a local sewer and its appurtenances, or a water main and its appurtenances suited to the requirements of such territory in the mode prescribed in Section 13 hereof, and the remainder of the cost and expense of such improvement shall be distributed and assessed against all the property within the bounds of said entire district in accordance with the special benefits conferred thereon and in proportion to area."

Under these local improvement laws, the City of Seattle and the City of Tacoma put in their distributing systems. Seattle uses only cast-iron pipe, and 8 in. as the minimum diameter. Tacoma uses 6 in. as a minimum diameter. The costs of all hydrants are assessed equally on the district.

Item 3.—Charges to the Municipality to be Paid from the General Funds.—Under this head will be:

(*a*) Charges to the fire department for the maintenance and operation of hydrants, and, where the cost of the system has been paid for out of water receipts, a charge should be made for overbuilding it from what would be necessary as a domestic supply to that of a "fire protection system". The only way to arrive at this proportion is to make a thorough study of the pressures needed, and the pipe sizes adequate for a domestic supply, and then place a value on the system thus estimated. Then calculate the cost of the water system necessary for an adequate fire protection, both as to pressure and supply. The difference in cost between a

Mr. Whitney.

system designed for adequate fire protection and one which is necessary only for a domestic supply, will be the amount on which the municipality should pay for interest, depreciation, and maintenance. Roughly, for a city of 50 000 people, the cost of fire protection service would constitute about 50% of the cost of the entire system.

(*b*) The school board should pay the cost of all water used in schools at the same rate as regular domestic consumers.

(*c*) The park board should pay, without any discount, the cost of all water used in parks.

(*d*) The board of public works should pay the cost of all water used for streets, sewers, or in any municipal department under its jurisdiction.

Item 4.—Charges to Consumers for Service Installations and Extensions.—In most cities a charge is made to a new consumer for putting in the house service and the meter. This is generally a fixed charge.

Item 5.—Charges to Consumers for Domestic Rate.—The domestic rate is generally ascertained, as a lump sum, by deciding that all other items shall be met out of the water revenue, and then adjusting the charges of the domestic and manufacturer's rates so that it will equal this amount.

Item 6.—Charges to Consumers for Manufacturing Purposes.—Items 5 and 6 are generally worked in conjunction with one another. After all else has been fixed, the revenue necessary to be derived from the domestic and manufacturer's rates can be determined. The only rational and satisfactory method of serving water to the public is by meter. Under a strict meter rate, it is known at all times how much water is being used and for what purposes. The Peoples Water Company, of California, serves the Cities of Oakland, Berkeley, Alameda, Richmond, Piedmont, and several small towns with water. The system is 100% metered. The writer had occasion to take from the books of this utility the quantities of water that were being used for various purposes in Berkeley during 1915, that city having about 12 000 service connections. It was found that, out of 100% of the live consumers, the distribution of the percentages were as shown by Schedule IV:

Schedule IV.—Percentages of Consumers Using Various Quantities of Water.

| | | |

Mr. Whitney.

The distribution of the various quantities is shown in Schedule V:

Schedule V.—Percentages of Quantities of Water Used.

Quantity used, up to and including	400 cu. ft.	15.0%
" " " " "	500 " "	22.7%
" " " " "	800 " "	42.0%
" " " " "	1 000 " "	50.4%
" " " " "	5 000 " "	81.6%
" " " " "	50 000 " "	93.0%
" " in quantities of more than	50 000 " "	7.0%

In setting rates in a municipality, it is often the desire of the community to have the water consumers pay a low rate, and do so, to a great extent, at the expense of the property owners. It often happens that there is, besides the municipal water department, a privately owned utility which serves the community with water, and, unless some provision is made to take care of the rates of the utility, in order that they may parallel those of the municipality, a discrimination will be worked on the privately owned system. In order to illustrate how to establish a low rate which will work fairly to both parties, the writer will assume a system designed for fire protection, the distributing system to be laid by local improvement districts, the collecting and transmission systems to be paid for out of the taxes or bonds, and the maintenance and operation, and emergency construction to be paid for out of the water receipts. The following data are assumed for illustration:

1.—Population	50 000
2.—Number of consumers	10 000
3.—Annual consumption, in cubic feet	200 000 000
4.—Percentage of water used per month less than 1 000 cu. ft.	23
5.—Percentage of consumers using less than 1 000 cu. ft. per month	54
6.—Percentage of water used per month, between 1 000 and 2 000 cu. ft.	27
7.—Percentage of water used per month, between 2 000 and 10 000 cu. ft.	31
8.—Percentage of water used per month, between 10 000 and 40 000 cu. ft.	9
9.—Percentage of water used per month, more than 40 000 cu. ft.	10
10.—Estimated present cost of system	$3 000 000
11.—Interest on bonds at 4½% per annum	$ 135 000
12.—Depreciation on system at 2%	$ 60 000
13.—Cost of operation and maintenance	$ 110 000

Mr. Whitney.

It is contemplated that the interest shall be paid out of the general tax on the system, and that the depreciation (either by sinking fund or otherwise) shall be paid from the water receipts, as will also the cost of operation and maintenance. This makes a total:

14—For depreciation, and maintenance and operation..	$170 000

Schedule VI.—Assumed Rates.

15.—For 1 000 cu. ft. per month or less.....	$1.00
16.—Cost per 100 cu. ft. from 1 000 to 2 000 cu. ft....	0.10
17.—Cost per 100 cu. ft. from 2 000 to 10 000 cu. ft....	0.7½
18.—Cost per 100 cu. ft. from 10 000 to 40 000 cu. ft..	0.5
19.—Cost per 100 cu. ft. for more than 40 000 cu. ft..	0.3

From Items 1 to 9 and 15 to 19, a table of rates (Table 5) has been compiled.

TABLE 5.—Rates of Return, as Indicated by Items 1 to 9 and 15 to 19.

54% of consumers at $1.00 per month (using 23% of the water per annum).....	$64 800
27% of 200 000 000 cu. ft. at 10 cents per 100 cu. ft. per annum.	64 000
31% " " " " 7½ " " " " " " "	46 500
9% " " " " 5 " " " " " " "	9 000
10% " " " " 3 " " " " " " "	6 000
Total revenue.....	$180 300

Table 5 shows that the revenue amounts to an annual rate of $18 per consumer. If it is decided to construct $100 000 worth of improvements to the distributing system and $200 000 worth to the collecting and transmission system, the distributing system to be improved by a local improvement district, bonds at 6%, and the collecting and transmission system to be improved with bonds against the municipality bearing interest at 5%, we have:

$100 000 at 6%.....	$6 000
200 000 " 5%.....	10 000

Making a total amount borne by the city, as follows:

$3 000 000 at 4½% (as shown by Item 11).....	$135 000
200 000 at 5%	10 000
Total municipal bond interest.....	$145 000
Local improvement bonds.....	6 000
Maintenance, operation, and depreciation.....	170 000
Grand total for support of system.....	$321 000

Mr. Whitney.

Inasmuch as the expense of the bonds is borne by the municipality, this leaves only the maintenance and operation to be taken care of through the water revenues. The estimated revenue, as shown by Table 5, is $180 300, and the maintenance and operation being $170 000, there is left a surplus of $10 300 for emergency construction.

Some method should be adopted whereby the privately owned utility can charge a rate which will be approximately the same as that of the municipality, and at the same time pay a reasonable revenue to its owners. If it is the desire of the municipality that the water consumers shall have low rates at the expense of the taxpayers, it apparently should not matter (if the utility is serving as good water at as ample a pressure and in sufficient quantities as the municipality) whether the difference is paid by taxes on bonds against the municipal system or by taxes on the general funds to subsidize the plant of the utility.

An annual budget equal to the interest on the estimated cost of the privately owned utilities plant could be assessed against the general taxes and allowed to this utility as rental for the use of its system and the privilege of using the fire hydrants; the utility should be allowed to pay—as mentioned in the case of the municipality—the maintenance, operation, and depreciation expenses, from the revenue derived from the sale of water, this revenue to be regulated by the city council or other authorized body. Thus competition would be placed on an equal basis.

The foregoing is a suggestion as to some of the ways in which revenue might be derived. Numerous modifications of these methods may be made, such as charging rentals for hydrants, street sprinkling, sewer flushing, etc. Inasmuch as all of these are paid for from monies derived from the general taxes, it amounts to the same in the end. Then, too, the revenues may be increased to the extent of having the department pay the expense of construction.

By the methods outlined herein a municipality can set a rate which will place as small or as large a burden on the water consumer as may be desired. The water must have a fixed cost, and what the consumer does not pay the taxpayer must.

Mr. Lippincott.

J. B. Lippincott,* M. Am. Soc. C. E. (by letter).—Almost without exception, the discussion brought forth by this paper agrees with the writer in that the service charges for a publicly owned public utility should be fixed on the same basis as those of a privately owned utility which is subject to rate regulation.

Mr. Harris has indicated the basic element when he says:

"It is only as a result of such an equitable attitude that private capital may be expected to 'pioneer' in those fields into which the public is not for the time desirous of entering."

* Los Angeles, Cal.

Mr. Lippincott.

It is not only that the present investment of private capital in public utilities should be protected, but that the policies adopted be such that the further investment in utility enterprises in undeveloped communities be fostered and encouraged by the assurance that these investments have the sanction and protection of legislative bodies. Mr. Yereance has also called attention to this point. Mr. Yereance and Mr. Thomson have discussed to some extent the relative merits and efficiencies of municipally operated utilities as against those privately operated, from which conclusions are drawn that it is difficult to obtain in municipally operated plants competent executives. It is generally not possible to pay salaries adequate to obtain the services of such men, and the executives are so hampered and tied with the red tape of municipal and political organizations that effective results are not generally obtained. Because of unreasonable abuse it is not a sufficient honor to hold a municipal office. Although this may be the general rule, yet some notable exceptions have come under the writer's observation. The Water Department of Los Angeles is most efficiently organized and operated. Santa Barbara and Long Beach, Cal., both own and operate their water-works. The management is efficient, and the earnings of the departments show a substantial profit.

Mr. Jordan's discussion is more fanciful than consistent. The relative merits of private or municipal ownership of public utilities is not at issue. The effort has been made in the paper to apportion the expense of a publicly owned utility so that each member of the community will carry his proportion and that private enterprise may not be discouraged. The segregation of the expenses is largely a matter of judgment.

Mr. Clark suggests that the consumers should pay through their rates only for the maintenance and operation of the system and the interest on the bonds, the depreciation being derived from the general tax levy. Relative to the redemption of bonds, Mr. Clark also states:

"If the city makes a 7% interest charge, as would be granted to a privately owned utility, any profit resulting under the bond rate of interest could be laid aside to redeem the bonds. If the bond rate of interest is 5%, the differential of 2% compounded annually at 4% will approximately retire the bonds in 30 years."

In the writer's classification of expense accounts, the effort has been made to create a service rate which would be in harmony with that for a privately owned utility. Under Mr. Clark's segregation of expenses, the service rates would be below those which could be profitably charged by a private corporation. Rates for privately owned utilities are fixed with a view of yielding the owner a fair profit on his investment. If this is done with a municipally owned plant the "fair profit" could be turned over to the general tax fund of the city. If the bond redemp-

Mr. Lippincott.

tion fund is paid from the general tax levy and the profits accruing from the operation of the system are paid into the general treasury of the city, it appears to the writer that the same end is achieved whether the bond redemption fund is carried by the utility department itself or by the city at large.

There is a surprising tendency constantly to put more burden on the property owner and less on the thriftless individual, whether that ownership be of realty or of a utility. The result tends to discourage investment in enterprise and to encourage indolence. Mr. Irving and Mr. Clark both consider that the owner of vacant property should be exempt from any special tax due to the utility other than that imposed on the city at large. It is the writer's contention that the benefits derived by the owner of vacant lots, because of the fact that the service of the utility is immediately available, is out of proportion to the charges on the property through taxes, and that, in addition to these charges, the owner should pay a special tax, probably on a front-foot basis. These fees should be considered as an addition to surplus, and could be turned into the general fund of the city. They have no counterpart in a privately owned utility, and would operate to the benefit of the community at large. Mr. Clark states: "It should not be the aim of a city to make its water consumers pay more than the cost of the service." It appears to the writer that a municipally owned utility should be viewed as a community investment and should be operated at a profit in the same way as a corporation is operated for the profit of its stockholders. The property owners are in effect stockholders in the municipal plant. Mr. Clark's contention concerning the depreciation of pipe lines in front of vacant property is true. Rate regulating bodies in California have adopted a practice of reducing the capital and depreciation accounts of private utilities where they appear to be overbuilt. Some such system might be adopted here to relieve the consumers from the depreciation charges for mains paralleling vacant property, and these charges might be made up from the proposed special tax on vacant lots.

Mr. Whitney's segregation of expenses is extremely interesting, and appears to the writer to be entirely logical. However, in the case of the competing private enterprise, it is doubtful if municipal legislative authorities would consent, possibly for political reasons, to the inclusion in the annual tax budget of an assessment to cover interest on the value of a privately owned plant.

Although it is not proposed to review the procedure of rate regulating commissions for private utilities, the writer might add, to what Mr. Hazen has said, that these commissions would increase their economic value to the community if they evolved a plan under which a premium was placed on efficient management of the utilities coming under their jurisdiction.

Mr. Lippincott.

In closing the writer wishes to express his appreciation of the discussion called out by the paper. He feels that the public welfare would be better served if policies were adopted by those in authority in legislative positions which would encourage and foster the further investment of private capital, particularly in those States which are in a formative or developing process. It should be kept well in mind that legislation or Court decree cannot compel the independent investor who is seeking new enterprises to enter fields where others already operating on such lines are losing money due to unfair competition.

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Paper No. 1390

CONSTRUCTION METHODS FOR ROGERS PASS TUNNEL*

By A. C. Dennis, M. Am. Soc. C. E.

With Discussion by Messrs. E. Lauchli, Robert A. Shailer, R. H. Keays, F. Lavis, James F. Sanborn, Lazarus White, T. Kennard Thomson, Francis Lee Stuart, J. V. Davies, S. A. Knowles, R. E. Dougherty, C. R. Hulsart, J. G. Sullivan, and A. C. Dennis.

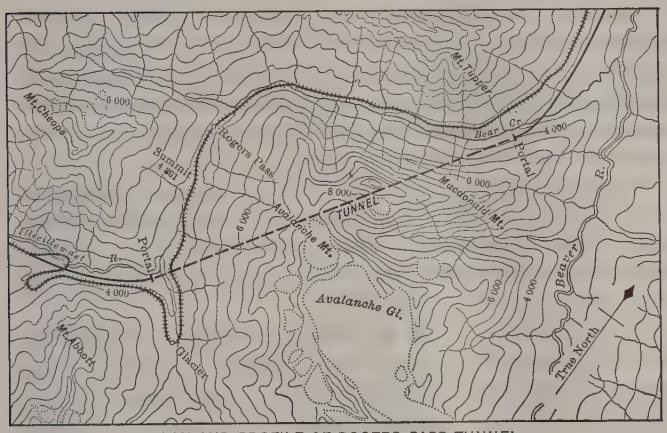
Synopsis.

This paper describes the construction of the Rogers Pass Tunnel, on the Canadian Pacific Railway, through the Selkirk Range, in British Columbia.

The principal feature of the paper is the description of the pioneer, or auxiliary heading, method of construction, by which: adequate ventilation of the headings was secured; the removal of the muck was facilitated; the water, air, and ventilation pipes were not subject to disturbance; it was possible to expedite the work of enlargement of the main heading; a route for men, tools, and materials was provided; and the ventilation pipes were carried around the drilling, blasting, and mucking operations in the main tunnel, thus preventing delays due to gas and smoke. The "pioneer heading" appears to have justified its use for this tunnel.

The Rogers Pass Tunnel is a double-track tunnel, slightly more than 5 miles long, being part of a local improvement, about 10 miles long, of the Canadian Pacific Railway Company's main line through the Selkirk Range, in British Columbia. The map and profile are shown on Fig. 1.

* Presented at the meeting of February 7th, 1917.



PLAN AND PROFILE OF ROGERS PASS TUNNEL

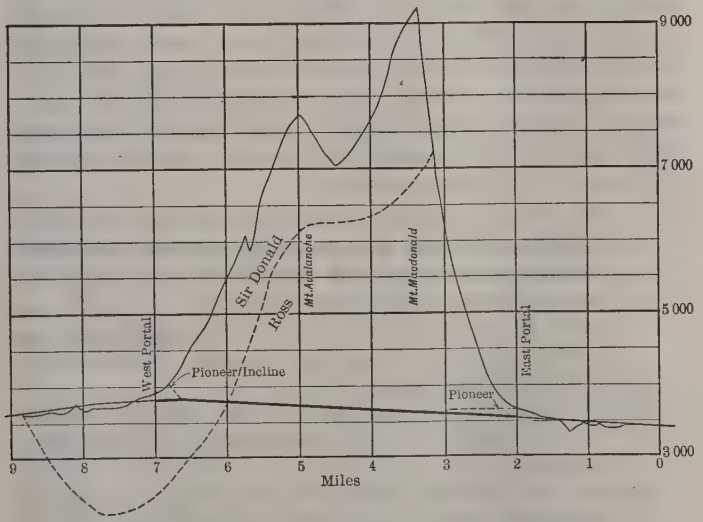


FIG. 1.

The writer visited the site of this tunnel in the fall of 1912, at the request of the vice-president of the Canadian Pacific Railway, reported favorably on the economics of the proposed improvement, and suggested to the chief engineer that a line be tried locating the east portal in Bear Creek Valley, instead of Beaver Creek Valley, as then proposed. This line was located and adopted between the time of asking for bids and of letting the contract.

The late V. G. Bogue, M. Am. Soc. C. E., reported favorably on this tunnel improvement in connection with a very able and thorough report to the Canadian Pacific Railway on improvements to its lines from Calgary to Vancouver, surveys and studies for which were then being made by Mr. F. F. Busteed.

The writer, knowing that the time of construction of this tunnel would weigh heavily with the Railway Company in considering bids, gave considerable thought to devising some quick method of construction. The usual American method of driving a top heading and taking out the bench with a power shovel was obviously too slow, and the ventilation would be too difficult, for a tunnel of this length. The European method, of a bottom heading and stoping out the rest of the section, would be too expensive, under existing labor costs. Shafts or adits were impracticable. It was finally concluded that a working tunnel or "Pioneer Heading" entirely outside of the regular tunnel section would be economically justified, under the existing conditions, for the following reasons:

1.—The pioneer heading would serve as an intake for forced circulation of fresh air through the cross-cuts and out of the main tunnel, enabling the work to be resumed immediately after blasting the main tunnel enlargement.

2.—It would serve to take muck from the headings around the drillers, blasters, and from shovel operations in the enlargement of the main tunnel.

3.—It would serve to conduct water, air, and ventilation pipe lines to the headings, so as to be undisturbed by the enlargement workings.

4.—It would make it possible to drill the main heading for enlargement far ahead of, and without interference from or with, the main tunnel blasting or mucking for enlargement.

5.—It would supply a route for men, tools, and materials to pass to and from the headings and enlargement drilling at all times.

6.—It would carry the ventilation pipe around the main tunnel enlargement drilling, blasting, and mucking operations, so that the gas and smoke from the heading operations would not foul the enlargement operations.

It was thus possible to keep the heading progress, enlargement drilling, enlargement blasting, and mucking going continuously, without any interference with one another whatever. Fig. 2 shows the relative location of these different operations.

Bids were asked for this tunnel work early in 1913, and the contract was let to Foley Brothers, Welch and Stewart, Railway Contractors, in June, 1913. This firm was not the lowest bidder, but undertook to do the work more quickly than other firms, its estimate being based on the proposed pioneer method, Mr. Stewart, of the contracting firm, and John G. Sullivan, M. Am. Soc. C. E., Chief Engineer of the Canadian Pacific Railway, being willing to take the risk of this untried method. The first study contemplated a top heading, but, at Mr. Sullivan's suggestion, this was changed to a location midway in the full section, so that the heading could be made smaller and still permit holes to be drilled far enough to break to the full section; otherwise, the work has been done as planned by the writer, and under his superintendence.

General Conditions.—The Selkirk Mountains in the vicinity of Rogers Pass are very rough, and are heavily timbered, except where the numerous snowslides occur. The rainfall is very heavy, and the snowfall is from 30 to 50 ft. The minimum temperature since work started has been — 32° Fahr., at an elevation of 3 800 ft.

Geology.

The approximate contact between the Ross and Sir Donald Series, as named by the Dominion Geological Survey, is shown in Fig. 1. The axis of this syncline is west of the West Portal, and the tunnel is through the eastern limb. The tunnel, except for 1 200 ft. of the east end and 400 ft. of the west end, is all in solid rock, classified as quartzite in the geological reports, but consisting largely of schists. The rocks contain no fossils, and cannot be assigned definitely to any

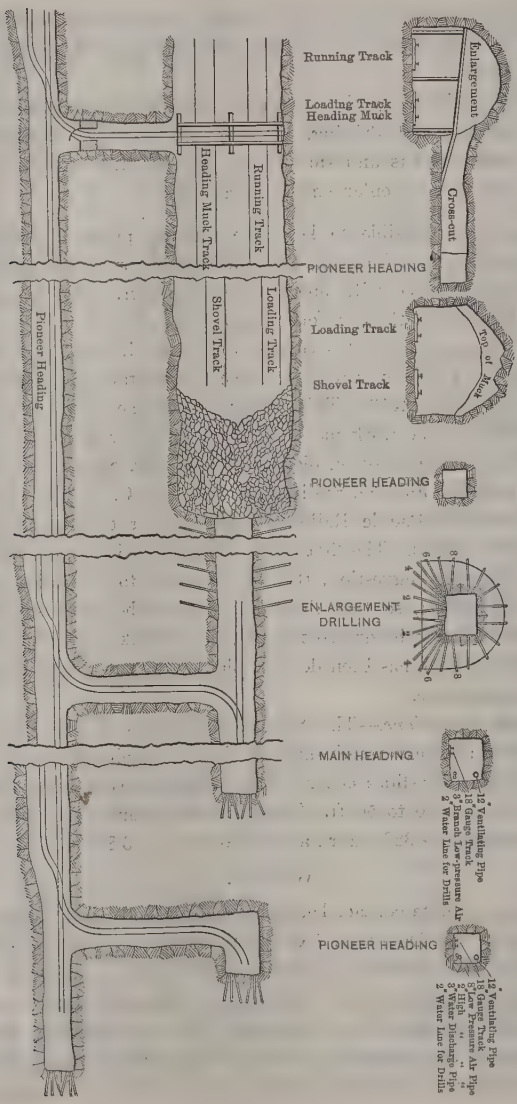

Running Track
Loading Track
Heading Muck
Heading Muck Track
Running Track
PIONEER HEADING
Pioneer Heading
Loading Track
Shovel Track
Loading Track
Shovel Track
PIONEER HEADING
ENLARGEMENT DRILLING
MAIN HEADING
PIONEER HEADING
Enlargement
Cross-cut
Top of Muck
12" Ventilating Pipe
18" Gauge Track
3" Branch Low-pressure Air
2" Water Line for Drills
12" Ventilating Pipe
18" Gauge Track
3" Low Pressure Air Pipe
3" High Pressure Air Pipe
2" Water Line for Drills

FIG. 2.

period, but appear to overlie the Upper Cambrian. The pitch is rather steeply to the west at both ends, and about level or pitching slightly to the east in places near the center of the tunnel. The strike is about 60° from the terminal axis. There are also many local irregularities and some faulting. None of the rock was timbered, except where broken up by faulting. Much of the tunnel that is in hard rock has slips, some of them clayey and talcy, which make the roof unsafe when weathered, and will require concreting. There was little water, owing to the fact that cracks which carried water formerly had become filled with quartz. The rock temperatures did not exceed 75 degrees. "Popping" was observed in the quartzite; that is, pieces of rock from the roof and sides, not shaken by blasting, would fly off with a loud report. This has always stopped after a few weeks exposure to the air.

Preliminary Work.

On the east side it was necessary to build 3 miles of temporary, heavy, side-hill railway to reach the camp site, and take out a rather heavy approach cut, before tunneling could be started. The approach cut was advanced sufficiently to start one wall-plate drift in January, and the other in February, 1914, the cut being finished in July.

On the west side about 1½ miles of temporary railway had to be built, and the same quantity of stream diversion, before the approach cut could be started. One 85-ton and one 100-ton shovel were worked both day and night in this approach cut, from December, 1913, until the finish, in June, 1914, when the tunnel drifts were started.

Camps.—The camps were built as soon as the track reached the respective sites, being finished about the end of 1913. They provided room for 500 men at each end, and were comfortable and sanitary. Hot and cold water, steam heat, sewerage, electric light, and good ventilation were provided.

Plant.—There was a power plant at each end, with five 150-h. p. boilers, equipped with special grates and induced draft, so that slack coal could be used for fuel. The boilers were run generally much beyond their nominal rating.

Each end had a capacity of about 4 000 cu. ft. of low-pressure air, that is, air at from 100 to 125 lb. at the compressor, with from 90 to 100 lb. at the drills, and a capacity of 1 500 cu. ft. of high-pressure air, that is, from 1 000 to 1 100 lb. pressure at the compressors, and

from 800 to 900 lb. pressure at the charging stations. Ingersoll-Rand compressors were used, two-stage for low and four-stage for high pressure, with inter-coolers. The steam ends were compound condensing. The compressed air passed through an efficient after-cooler, so that little trouble was caused by the freezing of moisture in the compressed air when exhausted by drills, pumps, and blowers. The plant was properly put in and properly looked after, and caused practically no delay.

Earth Section at East End.

There are 1 200 ft. of glacial drift at the east end of the tunnel. This material carries considerable water, due to seepage from the side of the mountain on one hand and Bear Creek on the other, the tunnel being below the creek level. The material was hard and compact when first opened, but ran when worked up or exposed to the air.

Wall-plate drifts (see Fig. 3) were started in January and February, 1914, as soon as the approach cut was down to the wall-plate grade, and finished to rock in July. From these drifts a stope was started in March, and segment timbers were placed for about 100 ft. Owing to a change of plan by the Railway Company, this segment timber had to be removed and rebuilt to a larger section, so that stoping and timbering ahead of this point could not be resumed until June. The timbering from the wall-plate up was completed in October. A locomotive crane was found very useful for taking the timber and other material from the framing yard and delivering it in the wall-plate drifts. After the wall-plate drifts met the main heading, in July, the material from the break-ups or stopes was hauled to the first cross-cut and out of the pioneer heading, so that it would not interfere with the timbering. Previous to that time it was dumped at the portal and reloaded by the steam shovel, together with the material of the approach cut. The material from the break-ups was passed down through a chute into the small tunnel cars. Considerable turning and crushing of the wall-plate occurred, necessitating a great deal of propping of the segments.

In August, the steam shovel, having finished the approach cut, started on the tunnel excavation, being operated by compressed air. During the day shift, the wall-plate was supported on temporary batter posts as the shovel progressed, and the night shift straightened the sides and set the plumb posts and lagging. During the night shift,

METHOD OF SOFT GROUND TUNNELING

Fig. 3.

the locomotive crane delivered the material required for the segment timbering the next day. After being impressed by the crushing of the wall-plate and segment timbers, it was with considerable anxiety that the undermining of the wall-plate with the power shovel was attempted. The characteristic of the ground, of remaining firm for a short time after being opened up, was encouraging. As it turned out, the fear of loosening part of the tunnel by the side sliding in, while the wall-plate was carried on batter posts, was groundless. There was no settlement of the wall-plate of more than 1 in., and the work was hastened and cheapened by this method much more than by driving drifts to place posts before excavating the core of the tunnel. The excavation of this earth section was finished in December, 1914.

Earth Section at West End.

There were about 400 ft. of soft ground at the west end. The west portal was under a swamp, difficult to drain, and at the intersection of two creeks which have both been diverted; but this soft ground section was at all times saturated with water, in spite of efforts at surface draining. The material was very fine sand and clay, with large boulders, and closely resembled quicksand or wet concrete. One boulder was large enough to drive the crown drift through. The cracks between the breast boards or poling had to be stopped with hay in order to prevent the material from running into the heading. This material was very heavy, and the small drifts had to have sets about 2 ft. apart, and additional posts under many of the caps. Boulders near the sides were jacked back into the quicksand in order to clear the timbers. The method of opening up this end was entirely different from that at the east end, as shown by Fig. 3. The bottom heading of the drifts for the plumb posts was started first, with a view to drainage, and was followed closely by the top heading. The muck from the top heading was trapped into cars in the bottom heading.

The crown drift was driven about even with the top heading of the plumb-post drifts, and seemed to be the most difficult of all to hold, due to the disturbance of the ground by the other drifts. When the plumb-post drifts had been driven to where the rock reached the level of the top of the plumb posts, the latter were discontinued and replaced for the remainder of the distance by wall-plate drifts, from

which a cross-cut was made to the main heading when the full rock section was reached.

After the plumb-post drifts were finished, the plumb posts, being double 12 by 16-in. timbers, were all set and thoroughly blocked against the timbering in the drifts, and the wall-plate was set on the rock beyond the posts. A break-out was started in the sides of the crown drift, about 100 ft. from the portal, and the timber segments were put in, working both ways from this break-out. The muck from break-outs was used to back-fill the post drifts. The posts and segments were set leaning away from the portal, and iron bars were spiked on the segments in order to resist the thrust of the material toward the portal. Other break-outs were started farther in, and segments were set under the crown bars, trued up, and packed thoroughly by timber between the segments and the timber supports of the outside segments. No wall-plates were used, and the segments were generally 32 in. thick. Work in this soft ground section was very slow and expensive. The time required for the 400 ft. was from June, 1914, to February, 1915. The excavation with the power shovel followed the timbering operations. There was some uncertainty as to whether or not bracing would be necessary to prevent the side pressure on the plumb posts from shoving them in after the excavation had removed the earth resistance on the tunnel side of these posts; but no movement occurred, and no bracing was needed. The material sets or hardens when drained.

East Pioneer Heading.

The east pioneer heading was started in September, 1913, about 50 ft. north of the main tunnel, 700 ft. west of the east portal, and about 60 ft. above the main tunnel level. This location was adopted in order to save 700 ft. of pioneer tunneling, to reduce the quantity of soft ground heading, to enable work on the heading to start sooner than that on the approach cut, and to get rid of the muck readily. The power was furnished by the temporary erection of an old compressor along the Canadian Pacific Railway track above and a pipe line down the hill to the work. This heading was run as nearly level as drainage would permit. The grade of the main heading reached the grade of the pioneer at the third cross-cut, the two former cross-cuts being driven to the dip, and material from the main heading being hoisted up the incline. The heading reached solid rock about 600 ft. in,

at which point the first inclined cross-cut was started, at about the beginning of 1914. The pioneer tunnel, in rock, was driven about 2 miles in 1½ years. The maximum monthly progress was 776 ft. The daily average was 20 ft. for the entire drift in rock.

West Pioneer Tunnel.

The west pioneer heading was started by an incline, 300 ft. long, from the rock outcrop, 700 ft. east of the west portal, about 150 ft. above the main heading level, and 50 ft. south of the main tunnel line. This location was selected in order to provide dumping ground, shorten the length of heading to be driven, avoid soft ground tunneling, and permit an earlier beginning than by waiting for the approach cut excavation. This incline was very wet, and took 2 months to drive, being finished in the latter part of July, 1914. This pioneer tunnel was driven for a length of more than 1½ miles in less than a year, the maximum monthly progress being 932 ft., which is far beyond any previous American record known to the writer. The daily average of 24 ft. for nearly a year, largely through very hard quartzite, is also unusual.

Pioneer Headings in General.

The pioneer tunnel, in rock, was 7 ft. high and 8 ft. wide. It was driven with light hammer drills, using hollow steel, with water attachments. Three drills, in general, but four in the hardest rock, were used in a heading. Spare drill machines, for the replacement of drills out of order, were kept conveniently at hand in the heading. No repairs were made under ground. The hammer drills are convenient and rapid, the delay and expense of their constant breakage perhaps balancing the advantage of speed under ordinary conditions. The drills are mounted on a light horizontal bar, about 18 in. below the roof line. Air and water are taken over the muck pile, or on hooks in the side, by a single hose line for each, to a manifold from which short individual hose lines supply the drills.

Light cars (½ cu. yd.) were used for muck, and the latter was taken off the track, instead of building sidings for this purpose. Shoveling plates were used at the face and on the side away from the track for some distance back of the face, in order to facilitate the handling of empty muck cars. The ventilating pipe was a 12-in. wooden water pipe connected to the Connersville blowers used for the exhaust.

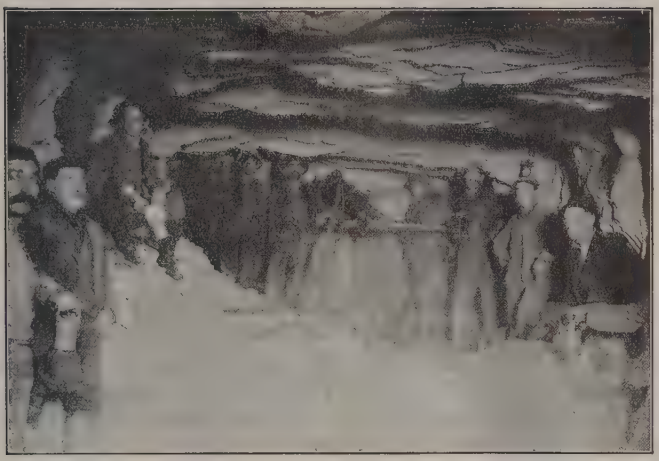


FIG. 4.—BEGINNING DRILLING FROM UPPER SET-UP OF BAR IN THE PIONEER HEADING.

FIG. 5.—MAIN HEADING, SHOWING ENLARGEMENT DRILLING.

This pipe was hung on the side away from the track, close up to the roof, and was carried to within 20 ft. of the face. Little damage was done to this pipe by blasting. The blowers were started exhausting when the first shot was fired, or a little before, and were run for 20 min. The men got back to work in from 5 to 10 min. No compressed air was allowed to be blown out for ventilating purposes. After a round was shot, the drillers followed the smoke back, barring down the roof, bringing explosives to re-shoot, and wetting down the muck pile, sides, roof, and face with water hose. The muckers cleared the track and began loading the muck which was scattered back.

When no further blasting was required, the lights were hung, the foreman sighted the line and grade point in the face, and the drilling gang set up the horizontal bar, placed their drills and proceeded. There was rarely any muck to be handled before the drilling could be started, as it was thrown back from the face by the heavy loading in the bottom holes and the fact that they were shot last, for this purpose. There were two helpers to three drills, and they brought up and changed the steel and adjusted the drill machines. When the drilling from the upper set-up was completed, the drillers took down the machines and carried them back, with the hose connections still attached, and oiled them up. After the mucking was done, the bar was dropped to the lower set-up, near the floor, and the drills were set to drill the bottom holes or lifters. The drills were carried forward, put on the bar, and were drilling sometimes in less than 2 min. after the bar was dropped. While the bottom holes were being drilled, the muckers laid the track, adjusted and covered the mucking sheets with muck, and brought up the explosives. The holes were loaded by the machine men, helpers, and foremen.

For the small part of the tunnel where re-shooting was not necessary, an 8-hour shift could do two rounds per shift, or a little better. Two men pick down the muck, and three men load the car and push it out, while three others stand by with an empty car, ready to put it on the track and load it. The three men taking out the loaded car return near the face with an empty car, take it off the track, and rest until the load comes out. The men get a rest from the monotony of steady continuous shoveling, and the empty car is available at once after the load goes back. The pipes for ventilating, and

for air and water were laid by a pipe man and helper, who looked after several headings.

Doing this work with muckers was unsatisfactory. Muck cars were taken from the heading back to a siding by a single mule, and from there to the dump by a two- or three-mule team driven tandem, until this method became inadequate, and then compressed-air locomotive haulage was substituted for the long haul. The heading muck cars, after the shovel and switching track had cleared a cross-cut, were taken to the cross-cut, pulled up an inclined trestle by air hoist and cable, and dumped into standard-gauge cars, as shown by Fig. 2. The cross-cuts are from 1 500 to 2 000 ft. apart, as shown by Fig. 6. Air pressure was maintained at about 90 lb. at the drills, which required 125 lb. at the compressors toward the end of the work.

The rounds were usually 6 ft. The cut holes were generally shot once or twice, and the remainder of the cut was shot with the rest of the round. All shooting in headings was done with fuse. The explosives used were 40 and 60%, low-freezing gelatine, with No. 8 caps. The rock was hard to break, and the quantity of explosives was necessarily high. From 21 to 28 holes were drilled in the pioneer face. Change of shifts was made at the heading, the shift coming on taking the tools out of the hands of the shift finishing. Three shifts a day were worked every day in the year, except for one day at the east end, due to the burning of the fan house, and one day due to the breaking of the air main by a snowslide. The pioneer gang drove the cross-cuts between the pioneer and the main tunnel heading. The pioneer tunnel was not driven for the last mile, connection being made by the main heading only, which was all drilled up for enlargement before the enlargement blasting reached this section. The main heading work had to be completed before the enlargement blasting and mucking reached the last cross-cut, as it would have been impossible to maintain the air connections, or ventilate the main heading, after that time, so as to allow continuous work.

Main Heading.

The main heading, generally known on the work as the "Center Heading", was entirely through the rock section. It was 11 ft. wide and 9 ft. high, the center line being the same as that of the com-

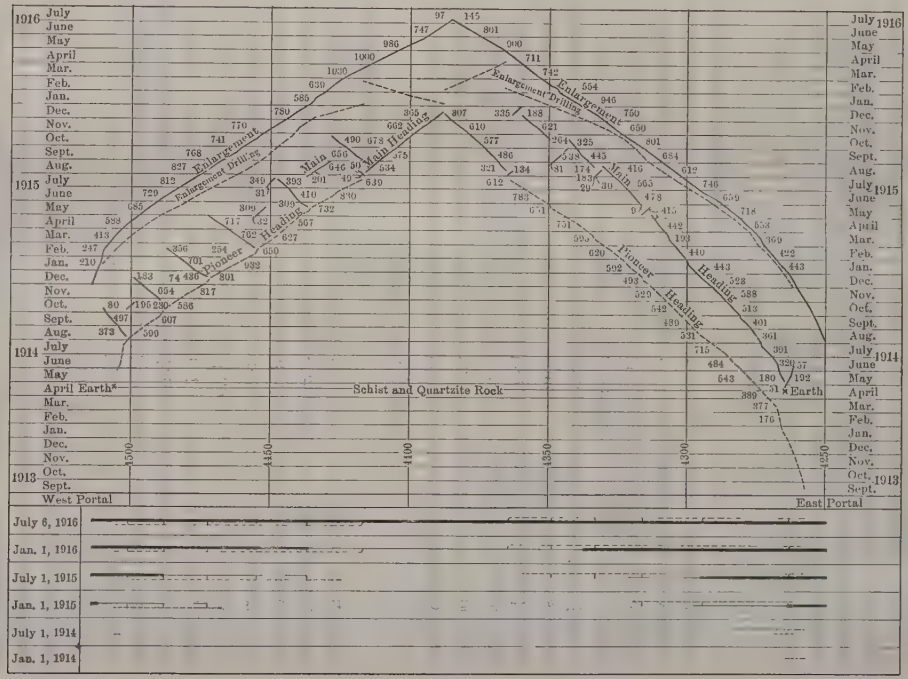

Schist and Quartzite Rock
Enlargement
Enlargement Drilling
Main Heading
Pioneer Heading
West Portal
East Portal
July 6, 1916
Jan. 1, 1916
July 1, 1915
Jan. 1, 1915
July 1, 1914
Jan. 1, 1914

FIG. 6.

pleted tunnel and the bottom being 6 ft. above the sub-grade. The position and size were such that lateral holes could be drilled from this heading to break the enlargement to the required dimensions. The air, water, and ventilating pipes for this heading were branches from the mains laid in the pioneer heading. Access to this heading was obtained through the cross-cuts from the pioneer, and muck was handled around the enlargement operations by the pioneer route. This heading was generally driven in a westward direction, on account of the drainage. The system of driving was similar to that in the pioneer. The rounds averaged about 7 ft., and 32 holes were drilled in the hardest rock. The main heading was sometimes driven from several faces. The progress is shown by Fig. 6. The average daily progress per heading at the east end was slightly more than 16 ft., and the maximum monthly progress was 621 ft. The average daily progress per heading at the west end was 20 ft.; the maximum monthly progress was 762 ft.

Headings in General.

The headings were sublet at a price per foot and a bonus for more than 450 ft. per month, the sub-contractor furnishing the labor and explosives only. This arrangement proved unsatisfactory, and was discontinued in September, 1914. After this time a substantial bonus, based on the monthly footage and equated for hard rock, was given, and divided among all men directly connected with the heading driving, in proportion to their regular wages earned for the month. It was agreed that the rate of bonus would not be reduced. The latter arrangement resulted in 23% greater speed, and a large saving in compressed air and other items furnished to the sub-contractor under the former arrangement.

Enlargement.

Drilling.—The enlargement drilling, after some experimenting, was done as shown by Figs. 2 and 7. Each hole was pointed by clinometer, the column carrying the drill being set always at the same distance off the center line, and the arm for the lower and upper sets being always the same distance above the sub-grade. Line and levels were furnished by the Railway Company's engineers, and a string was stretched by which the columns and arms were located. Each drill hole had its proper distance from the arm. The drill

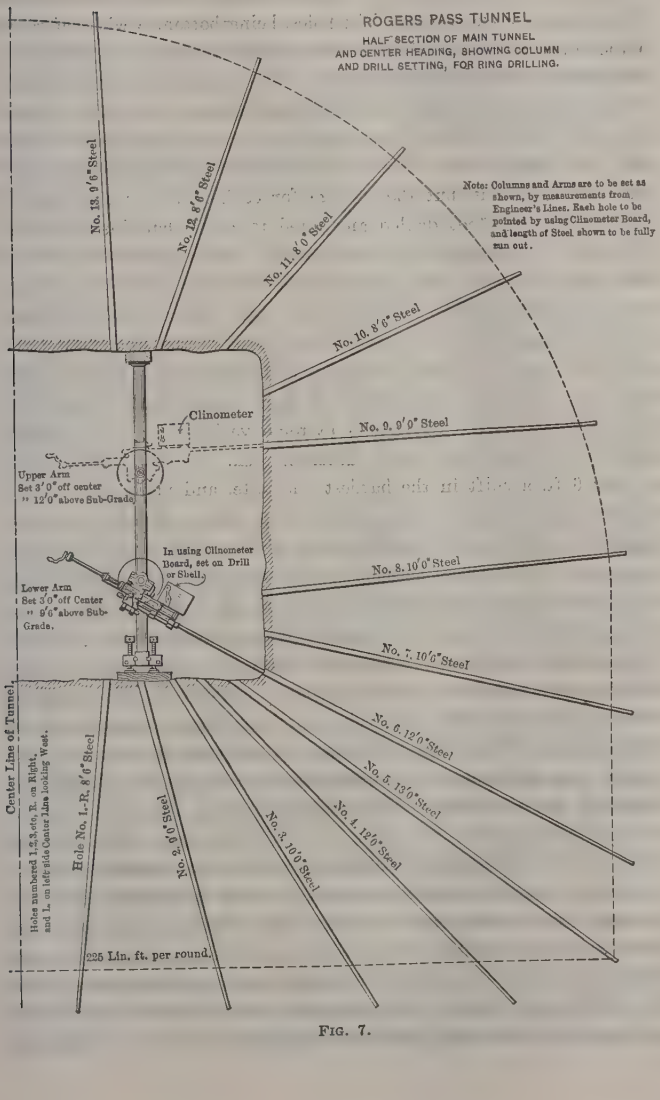

ROGERS PASS TUNNEL
HALF SECTION OF MAIN TUNNEL
AND CENTER HEADING, SHOWING COLUMN
AND DRILL SETTING, FOR RING DRILLING.
Note: Columns and Arms are to be set as shown, by measurements from Engineer's Lines. Each hole to be pointed by using Clinometer Board, and length of Steel shown to be fully run out.
No. 13. 9'6" Steel
No. 12. 8'6" Steel
No. 11. 8'0" Steel
No. 10. 8'6" Steel
No. 9. 9'0" Steel
No. 8. 10'0" Steel
No. 7. 10'6" Steel
No. 6. 12'0" Steel
No. 5. 13'6" Steel
No. 4. 13'6" Steel
No. 3. 10'6" Steel
No. 2. 9'0" Steel
Hole No. 1.-R. 8'6" Steel
Clinometer
Upper Arm
Set 3'0" off center
" 12'0" above Sub-Grade
Lower Arm
Set 3'0" off Center
" 9'6" above Sub-Grade.
In using Clinometer Board, set on Drill or Shell.
Center Line of Tunnel.
Holes numbered 1, 2, 3, etc., R. on Right, and L. on left side Center Line looking West.
225 Lin. ft. per round.

FIG. 7.

holes were thus bottomed at a regular distance beyond the neat line of the completed excavation. The holes, being bottomed with reference to the line and grades given by the engineers, were not affected by irregularities in the heading driving. The columns were set by men for that purpose, so that the drillers and helpers had only to do the drilling. The drill steel was brought to the drillers, and the dull steel was taken away. The drillers and helpers were paid their wages in any event, but the footage for each man was kept, and if the price set per foot drilled amounted to more than his wages, he was given the difference as a bonus check. Air and water connections were made for every third ring of holes, and only one drill machine, though handled by each runner of the three daily shifts, completed the three rings, and then moved to the head of the line, taking the next three rings. Congestion of men and material was thus avoided, and each man had a fair chance to work on an equal quantity of hard and soft rock. There was extreme variation in the quantity drilled by different men and in different rock. The same man might do only 6 ft. a shift in the hardest quartzite, and more than 100 ft. per shift in the softer schist. New men, after a month's practice, generally made more footage than men of long experience in mining. In general, it was found better to train green men than to try to get men accustomed to piston drills to learn to run hammer drills. Most of the rings were 6 or 6½ ft. apart. When explosives rose in price it was found economical to space the rings 5 ft. apart, as the extra drilling cost was balanced by the saving in explosives, with the added advantage that the muck was broken into smaller pieces and scattered farther back. Where the roof was soft and full of slips, so that trouble was anticipated, the upper set of arms on the column was lowered 1 ft., in order to leave some trimming of the roof to be done by jack-hammer, flat holes, and light blasting. The air and water for the enlargement drilling, as well as the supplies, came by the pioneer tunnel and the cross-cuts, so that this drilling was not disturbed by the enlargement blasting. The drilling for the last mile, where no pioneer tunnel was driven, was started at the middle and progressed toward the portal, the track, pipe, etc., being removed as the drilling was finished. The stopping of the pioneer tunnel was well-timed, as the main heading was driven and the enlargement

drilling completed just in time to avoid delaying the enlargement blasting and mucking at the east end.

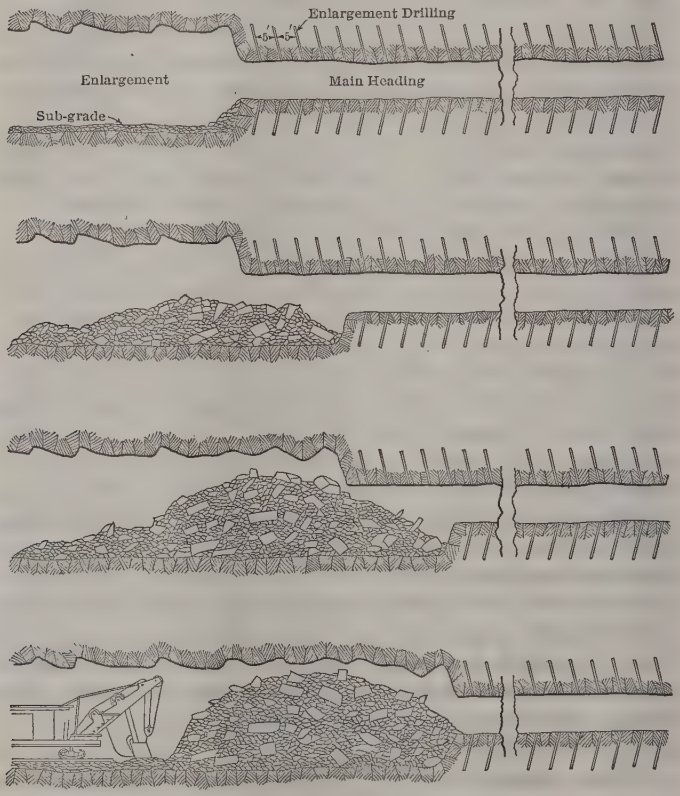
Enlargement Blasting.—There was considerable difficulty in breaking the bottom to sub-grade when the rock excavation was first started. This was overcome by dropping the floor of the main heading 1 ft. and drilling the holes in the bottom 1 ft. deeper. Difficulty was found also in getting the sides below the springing line to break for the full width. This was overcome by drilling one or two relief holes at this locality in tough breaking rock. In tough rock, two, four, six, and sometimes eight, holes were sprung. (Fig. 2.) If oversprung, the ring being shot was likely to break into the next ring and explode it, or shake it up so as to spoil the effect of the blast.

Generally, from ten to fifteen rings were kept loaded ahead. Any part of a hole which had not broken, and could be found, was reloaded and shot with the next ring. Generally, a little muck was left in the face by the power shovel in order to prevent the first ring from scattering back too far. If the previously shot material had not broken to the required width, however, all the muck was loaded, and jackhammers were used to drill up this tight rock, after which it was shot before the regular rings were blasted. Several bottom rings were first blasted, then a top and bottom ring were blasted together until the muck piled up to within 4 or 5 ft. of the roof. Then blasting was discontinued, and the men scaled and trimmed the roof, working from the muck pile. (Fig. 8.) Where no holes had to be reloaded, rings could be blasted at intervals of from 15 to 20 min. The blasting was done with a battery in the main heading, and the bottom holes were all loaded ahead, the wires being wound up and stuck in the holes, from which they could readily be pulled out and connected. The upper holes were loaded, but no primers were put in until ready to blast. The holes were loaded to within 4 ft. of the collar, whether sprung or otherwise.

When retiring in the main heading to blast, the blasting gang took back the scaling tools, so that they might examine and scale the roof of the heading if necessary as they returned. After several rings had been blasted, the power shovel crew commenced to clean up the beginning of the muck heap, and only retired a few minutes for the following blasts. Several top rings were generally held and shot at meal times, when the shovel had excavated sufficient muck to

provide room for more without blocking the airway and manway over the pile.

The smoke and gas from the blasting were quickly taken out by the fresh air forced into the pioneer tunnel by a "Sirocco fan" at the



LONGITUDINAL SECTION, SHOWING ENLARGEMENT BLASTING

FIG. 8.

portal. The air circulated through the pioneer tunnel and the cross-cut ahead of the blasting, and then back through the main heading, over the muck pile, and out at the portal of the main tunnel. This circulation was prevented from short circuiting by stoppings and doors in the

cross-cuts passed by the shovel. The quantity possible to shoot depended on the distance the muck was thrown back, or the quantity of muck for which there was room without interfering with the scaling and trimming of the roof; it varied from 24 to 110 ft. Much time was lost in taking up the track before blasting, in cleaning up the thinly scattered muck directly after a blast, and by other delays incidental to each clean up, which made it desirable to shoot as many rings as possible.

Enlargement Mucking.—Mucking was done with small steam shovels, with cylinders enlarged so as to be efficient at 100 lb. air pressure, and with the boom sheaves set back so as to shorten the booms and protect the sheaves. The shovels loaded the muck into 12-yd., standard-gauge, air-dump cars. The cars were shifted to the shovel by two small compressed-air locomotives, and were taken from a spur near the shovel to the portal by a larger compressed-air locomotive; they were taken by a steam locomotive from the portal to the dump. During the past year the shovels mucked 18 550 ft. throughout 3½ miles of tunnel, or more than 2 ft. per hour. Blasting and trimming took about one-quarter of the time. The best monthly run for one shovel was 946 ft. at the east end, and 1 030 ft. at the west end. The ground between the third and fourth cross-cuts at the west end became so dangerous, owing to the material in the roof and sides falling, that one shift out of three had to be devoted to scaling this section until it was concreted. About 2 500 cu. yd. fell or was scaled in this section, on account of the disintegration of the material on exposure to the air. The scaling car throughout the work was handled by the air locomotive, and the scaling was done by the shovel crew or others during the time the shovel was stopped for enlargement blasting. Any rock not broken to the required dimensions was drilled off the muck pile, or from the floor, as the shovel cleaned up as far as possible, or from a car left at the shovel crew's meal time, and shot before the next rings were blasted. The enlargement drill holes were the general guide as to the trimming required, such points as were missed being marked by the Railway Company's engineer. There was very little over-breakage.

Concreting.

About 1½ miles of the tunnel, including the soft ground at each end, required concreting. This work was sublet to the Bates and

Rogers Construction Company, of Chicago and Spokane. The concrete section is heavily reinforced, which prevented the use of the pneumatic method of placing. The sub-contractors used wooden forms, and deposited the concrete from a platform near the roof reached by an inclined trestle. The concrete mixer was on a car, and the materials were on other cars back of it. The concrete from the mixer flowed into a small car which was hauled by cable up the trestle incline to the high platform, from which it was shoveled into the forms. Much of the lining required back as well as front forms, and the space behind the back forms was filled with rock or wood. This back-form and back-filling work was slow and expensive, especially where there were only a few inches between the back forms and the rock. The concreting has been well done, but the work is behind time, owing largely to conditions which the sub-contractors could not control.

General Results.

The tunnel has been finished 11 months ahead of the contract time, and for a substantial sum less than the price bid. There have been no strikes, or interruptions of the work. The management has had the necessary money and authority, and has been given a free hand, both by the Railway Company and the contractors, and has had the loyal support and valued assistance of the men, notably Messrs. J. Fowler, Assistant Superintendent, T. Truran, Master Mechanic, J. Roberts, Chief of Office Force, and M. C. Brian, Shovel Runner.

Engineering.

The Railway Company did the necessary engineering work. Messrs. F. F. Busteed, H. G. Barber, and W. A. James, M. Am. Soc. C. E., were successively in charge, Mr. J. G. Sullivan being Chief Engineer.

DISCUSSION

Mr. Lauchli.

E. LAUCHLI,* ESQ.—Those who have followed closely the progress made in driving Rogers Pass Tunnel cannot but admire the skill displayed by the engineers and contractors connected with this great enterprise; and, no doubt, the completion of this bore will give a greater impulse to work of this character in America.

The speaker, who was retained in an advisory capacity by one of the contractors bidding on this work, was well aware of the conditions preceding the driving of this tunnel. In his opinion, bidding on this work was a gambling proposition without precedent, owing to the fact that the railway company did not provide the bidders with sufficient information relating to the geological formation of the range to be tunneled. The accuracy of this statement is backed by the fact that, according to Mr. J. G. Sullivan, Chief Engineer of the Canadian Pacific Railway, expert tunnel contractors bid on this work from $8 to $11.25 per cu. yd. for excavation—prices hitherto unheard of in connection with double-track tunnel excavation.

That the driving of this tunnel has been a success will not be denied by the Profession, but, in the speaker's opinion, the results attained are due more to the unusually favorable conditions—as far as material penetrated, absence of rock pressure, small quantity of water underground, and low rock temperatures are concerned—than to the driving methods adopted by the contractors.

In preparing their bids, those interested in this work, had to assume that there would be a reasonable amount of difficulty, and this, no doubt, accounts for their selection of the bottom-heading method of tunneling, the only method known to-day, which insures absolute success, both under favorable and unfavorable conditions. Now, as just mentioned, there was no information available of a nature to cause bidders to select a driving method, practicable under ideal conditions only, and no experienced tunnel contractor could afford to gamble on an undertaking of this magnitude.

The driving method actually adopted based its greatest expectations on the enlargement of the tunnel, and necessitated an auxiliary or pioneer tunnel. Ventilation alone did not call for an exceptional ventilation system, as has been proved during past years, in connection with the driving of tunnels from 5 to 9 miles long, driven under far less favorable conditions than those at Rogers Pass.

Had an intense ground pressure developed over a long section, or over several short sections, then in order to keep within the schedule of progress, the method of enlarging the bore at one point only would have necessitated such radical changes as to jeopardize the success of the enterprise. In Rogers Pass Tunnel it was possible to enlarge

* New York City.

Mr. Lauchli.

the tunnel at one point only, and an obstacle, such as intense ground pressure at the enlargement point, would have necessitated keeping the timbering, and probably also the masonry lining, close to the enlargement; and thus the blasting and mucking operations would have been greatly hampered.

On the other hand, the method of enlarging the bore to full section from several working points, as used for instance in the Loetschberg, Hauenstein, and Grangs Tunnels, in Switzerland, possesses the advantage of being adaptable both in light and heavy ground, and, under favorable conditions, such as met in Rogers Pass Tunnel, the drilling and blasting operations are at least as economical as those used in the last bore referred to, the only differences being the temporary heading timbering and the removal of the two side benches adjacent to the heading.

It may be useless to point out here that the removal of the benches could have been done with air-operated shovels. It is questionable, also, whether the total cost of driving this tunnel by the method actually adopted was less than it would have been by using the bottom-heading method. In the first place, the pioneer or auxiliary tunnel method involves practically two headings instead of only one, as used in connection with the European method; for, in this case, the length of the pioneer tunnel, together with the cross-cuts, is a little more than 20 000 ft., and, of course, this auxiliary tunnel is chargeable to the main bore. Then, also, it is questionable whether the weathering or ground pressure will not incur maintenance charges in connection with the pioneer tunnel and the cross-cuts, in order to avoid disturbances of the rock surrounding the main bore. The practice, also, of not lining the main bore at the very outset is open to criticism, as it is always desirable to prevent initial disturbances from taking place in the overlying strata, especially in deeply overlaid tunnels, rather than to check these later, perhaps under operating conditions in the tunnel.

The speaker, who made a careful study of the proposition, figured that, on the assumption of a daily progress of 20 ft. from each portal, and assuming that one-third of the tunnel would necessitate temporary timbering, also that the masonry lining would follow close to the excavation, the cost of driving this bore, with a bottom heading, would be $5 per cu. yd., exclusive of contractor's profit. Mr. J. G. Sullivan has stated* that the cost of driving the tunnel through rock, including the cost of driving the pioneer tunnel and the cross-cuts, "will amount to a little less than $5 per cu. yd. for excavation in the tunnel proper."

In comparing the costs of different methods, the mistake has often been made of comparing short tunnels with long ones, such as the

* *Engineering News*, February 24th, 1916.

Simplon or the Loetschberg; for, it is a well-known fact, that the cost per linear foot of long tunnels is higher than that of short ones. For instance: for the first kilometer of the Loetschberg Tunnel, the cost was $160 per lin. ft. including lining; the cost for the seventh kilometer (center of tunnel) was $224 per lin. ft., the average for the whole bore being $201.65 per lin. ft. Mr. Lauchli.

On the other hand, the cost of Rogers Pass Tunnel can be compared favorably with that of the more recently completed Hauenstein Tunnel, 26 680 ft. long, driven under conditions very similar to those encountered in Rogers Pass Tunnel, although less favorable. The wages paid in connection with the Hauenstein Tunnel were higher than in the case of the Loetschberg Tunnel, being about half as much as those paid at Rogers Pass Tunnel.

On the other hand, the tunnel was driven 20 000 ft. from the south portal, the remaining 6 680 ft. being driven from the north portal, and this alone increased the cost. Another feature of material importance is the fact that the bore was lined throughout, the masonry lining being kept close to the excavation. This feature also complicates driving operations, and increases the cost of excavation by about 50 cents per cu. yd.

Finally, the mining operations had to be carried on very carefully on account of swelling ground, and at several points masonry inverts had to be provided; yet the cost of excavation was less than $2.70 per cu. yd. Had it been possible to line this tunnel after the completion of the excavation, and, also, had the tunnel been taken out equally from each end, there is no doubt that the cost for excavation would not have been more than $2.25 per cu. yd.

This bore was driven at an average speed of 750 ft. per month for the heading, for a whole year (maximum 1 030 ft.), and at the rate of 658 ft. per month for stoping (maximum 870 ft.), also at a rate of 656 ft. per month for enlarging to full section (maximum 850 ft.); and the masonry lining was carried on at the rate of 650 ft. per month.

Robert A. Shailer,* M. Am. Soc. C. E.—The speaker is of the opinion that the pioneer heading, entirely outside the main tunnel section, is a new feature in tunnel construction. Mr. Shailer.

Apparently, the time of completion was vital, and the fact that the contract was completed more than 11 months ahead of the specified time surely shows that the adoption of the pioneer heading was justified economically.

To the advantages which Mr. Dennis claims for this heading there might be added: that it enabled the work on the long rock tunnel to be well under way before the earth and soft ground approaches on the

* New York City.

Mr. Shaller.

east and west ends were commenced; also, that the pioneer heading served the purpose of a pilot tunnel to inform the contractor as to the character of the rock through which the main tunnel was to be driven.

Had the speaker's opinion been asked as to the plant layout before the work was commenced, he would have said that five boilers, each of 150 h. p., would prove inadequate to fulfill the requirements at each end of the tunnel, namely: 4 000 ft. of free air compressed to 100 to 125 lb., and 1 500 ft. of free air compressed to 1 000 to 1 100 lb., as well as power for the lighting, ventilating, and pumping plants; and apparently, that opinion would have been—in some degree, at least—correct, for, the author says: "The boilers were run generally much beyond their nominal rating", and this, as all know, is not good for the boilers or for the contractor's pocketbook.

It is noted that there was a drop of approximately 15% from the power-house pressure to the drills, and of 25% to the high-pressure charging station, due probably to the pipe lines being of too small diameter. This loss seems excessive, and must have proved expensive.

The descriptions of the various tunnel operations are clear and concise, and the speaker thinks the members of the Society are indebted to Mr. Dennis for this paper.

Mr. Keays.

R. H. Keays,* Assoc. M. Am. Soc. C. E. (by letter).—This is a very interesting paper, as it explains new methods for obtaining speed in driving long tunnels in rock.

Some years ago it was generally stated that the art of tunnel driving in Europe was much further advanced than in America, as was shown by the great progress in the various tunnels through the Alps as compared with America's much more modest attainments. The proper reply was to argue that the reason American engineers did not specialize in speed was that they had no long tunnels to drive, and when, for financial reasons, they found it necessary to speed up their work they would soon find a way to do so. Certainly, some of their records of late years have been very creditable.

In the writer's opinion, a large part of the credit for the great progress should be given to the improved types of drills developed during the last few years. Where it used to be agreed that the controlling feature of speed was the time taken to drill the heading. it is now considered that tunnel driving is a mucking proposition, and the feature to which most attention should be devoted is the cleaning away of the muck.

In explaining why the pioneer tunnel method was used for the Rogers Pass Tunnel, the author states that the usual "top heading and

* Jersey City, N. J.

bench" method was considered too slow and the "bottom heading" method too expensive. Mr. Keays.

As to the "top heading and bench" method, the writer is of the impression that it has not yet shown its capabilities as to speed, no tunnel driving with modern drills having as yet been attempted where speed was the primary object and where the conditions were so nearly ideal for fast work as in the Rogers Pass Tunnel. The writer has never used the bottom heading method, but has always heard it recommended for its economy. Substantiating this claim, the writer knows of several tunnels where it was used, but they were of such short length that speed was of minor importance.

From a careful reading of the paper, the writer is of the opinion that the author has not demonstrated that the pioneer heading has enough advantages to justify its use as a standard method.

The author gives a list of reasons for using the pioneer heading, showing various advantages which no doubt would expedite work in the main tunnel; but, in the writer's opinion, it would be very little. The paragraphs explaining the location of the portals of the pioneer headings, however, would indicate that, on account of avoiding a lot of soft ground work, the pioneer heading could be driven far in advance of the main heading, but as no heading was driven in the main tunnel, using the pioneer heading to work from, the writer does not understand how greater speed was made in the main tunnel on this account. It merely cheapened the cost of driving the pioneer heading and provided better dumping ground.

The writer assumes that the main heading was being excavated at one face only at any one time, and also that enlargement operations were being carried on at one place only.

Of the reasons given by the author for adopting the pioneer heading, No. 2 is the only one that could tend to expedite work in the main tunnel. Reason No. 1, treating of ventilation, should have no influence on the speed, as the matter of ventilation at the working face can be taken care of very well in the "top heading and bench" method, and ventilation between the heading and portal is not important.

No. 3, of course, would not be an advantage in the "top heading and bench method", for the enlargement operations would be close to the heading.

No. 4 could hardly be called an advantage, for the rate of speed depends on the enlargement operations.

No. 5 is rather unimportant.

No. 6 would also be of no advantage, as the enlargement operations are close to the heading.

Ventilating systems, as usually put in, are disappointing. There is no agreement as to the relative advantages of the pressure and suc-

Mr. Keays.

tion systems, the size of pipe to be used, the pressures, or the quantity of air necessary.

It may be said that much larger pipes are required with suction than with pressure systems in order to handle the same quantity of air. The suction system keeps the air of the whole tunnel in relatively good condition, but, at the heading itself, where the men are working, it has the least effect. This trouble is augmented, generally, by the fact that the pipe used is of very light weight and is leaky at the joints, so that there may be very little air taken in at the end.

The pressure system, on the other hand, clears out the heading first, so that the men can go back to work immediately, and ventilation between the working face and the portal is not important.

The pressures generated at the fan are usually so low that a very large pipe is needed in order to get the volume. For economical reasons, a large pipe is always made of light material, which is not satisfactory.

The writer, therefore, on account of the foregoing considerations, believes that a medium pressure system, using a relatively small, screw-joint, standard steel pipe for delivering air to the heading, is the best.

On a recent job he used a Connersville blower, good for a pressure of 10 lb., delivering 750 ft. of air per min. to a heading somewhat more than a mile distant. The pipe was of steel, 6 in. in diameter, and the maximum pressure was about 7 lb. The condition of the air at the heading was always good. The heading and bench were always shot at the same time, every 8 hours, and the blower was usually run for about 2 hours each time.

Mr. Lavis.

F. Lavis,* M. Am. Soc. C. E. (by letter).—This account of the most successful tunneling operation in North America—and this is said with, it is believed, due appreciation of all the work of this kind which has been done in recent years—is a tribute to the advent of the engineer as a director of a type of construction which requires the planning and organizing ability of which usually only the trained engineer is capable.

Confronted with the problems of organization required for the successful economical completion of the long Alpine tunnels, Europeans long ago realized that these were problems for the engineer, and were not of the kind which could be put through solely by "pick, shovel, and pluck," or mere driving power, unaided by a trained mind to plan and execute.

The writer fully recognizes the value of the qualities of the contractor, especially those of the railway contractor, which are very necessary, and which engineers seldom possess. The need of planning,

* New York City.

of co-ordinating the work of the organization with the plans, the successful modification of the plans to fit both the conditions and the organization as they develop, are, however, essentially qualifications which the properly trained engineer alone possesses. Mr. Lavis.

Though this is not the first tunnel driven in North America which has demonstrated the ability of the engineer as an organizer along lines which require this necessary co-ordination of planning with execution, it is the latest and most successful, and the writer believes that the realization of this phase, of the conduct of the operations which have resulted so successfully, is of quite as great importance as the recognition of the successful application and improvement in the "pioneer heading" method, important as this latter is as a demonstration of efficiency.

There is little opportunity for comment in regard to the methods used. Figures are not given, though it is hoped they may be supplied later, either by the author or by the officials of the railroad company, showing the value of the time saved and the economy of the extra expense involved in the speed of driving. The writer has no doubt the expense was justified, but the author's comments would be of great value, if he would indicate his views on the economical speed from the standpoint of construction alone without regard to operating losses to the railway, and on the value of the saving to the railroad company.

There is a growing tendency to give, in connection with the description of works of this kind, either exact or approximate costs, and it is rather to be regretted that they have not been stated in this case, as such information has a distinct value, and its suppression is of doubtful utility to the railroad company or the contractors.

The paper is filled with instances which show the careful continuous study of conditions and improvements in details as the work progressed. Many of these latter are not altogether new; some of them have been advocated by the writer and others; but as their general application to tunneling is by no means common, even yet, it seems permissible, even at the risk of repetition, to refer briefly to some of them:

Ventilation is a *sine qua non.*
Bonus system used.
Hammer drills used entirely, columns carefully set with relation to line and grade, holes pointed by clinometer.
Three helpers were used for each two drills.
Fuse was used instead of batteries for blasting.
Water was used to wet down the muck pile and wash the dirt from the sides, roof, and face.
Muckers have brief rest between each carload.
Mucking sheets were used.

Mr. Lavis. There is also one statement full of significance to those who have had much tunnel experience, namely, that "The plant was properly put in and properly looked after, and caused practically no delay". This epitomizes the attitude of those responsible for the conduct of the work, and indicates one of the most important reasons for the success attained.

Mr. Sanborn. JAMES F. SANBORN,* ASSOC. M. AM. SOC. C. E.—The speaker has been asked to call attention to a volume, now being prepared regarding the tunnels of the Catskill Aqueduct, recently completed. Dr. Charles P. Berkey, of Columbia University, the Geologist of the Board of Water Supply, is undertaking to collect, as far as possible, information for a comprehensive history of the geological features which had an important bearing on the tunnel design and construction of the several long siphons and other structures of the Catskill Aqueduct.

It is hoped that this volume will bring out some of the finer points of tunneling. Usually, a tunnel is driven, and one never hears anything more about it, unless, like Mr. Dennis, somebody has the courage to describe it in a paper.

Mr. White. LAZARUS WHITE,* ASSOC. M. AM. SOC. C. E.—The speaker visited Rogers Pass Tunnel in September, 1915, when it was being driven at about its most rapid rate. The profile (Fig. 1), shows that the mountains are only 9 000 ft. high, but, to the speaker, they looked as though they might be 19 000 ft. It was raining at the time; the mountains were covered with snow and ice; and there was snow in the woods; but the tunnel was dry. The work had then been advanced more than 2 miles from the west end and there was just a little trickle of water coming out of the entrance.

The rock was "made to order". It was one solid block, as far as could be seen, practically without faults or the disturbances usually found in rock, and, to the speaker's unpracticed eye, it appeared to be a hard shale or slate. He would hardly classify it as a hard quartzite, as Mr. Dennis does; but he probably knows.

The pioneer heading method appeared to be a very good one for that place, and the whole scheme of the tunnel and its execution a very good piece of work. In the speaker's opinion, it was the first successful combination of the Swiss and the American methods of tunneling. Other attempts have been made to combine the two systems, but they have not been successful, either financially or otherwise. This project, however, was successful in all respects.

The pioneer tunnel was of quite a small bore, and, as Mr. Dennis has stated, it served primarily for ventilation, because through it any quantity of air was sent around the heading, and blew out the smoke. The tunnel was both the driest and the best ventilated of

* New York City.

any the speaker has visited. On account of the extensive blasting that was done, it must be realized that the ventilation had to be well nigh perfect, because from fifteen to twenty rounds were set off in succession in order to break up the rock ahead of the steam shovel. Mr. White.

In any ordinary method of tunnel ventilation, this would have made the work difficult, if not impracticable, but, with this pioneer tunnel, the air was sent around and blown through the main tunnel. There was an immense fan in the pioneer heading through which there was always a strong wind blowing.

The most original part of the pioneer heading has a strong resemblance to the double tunnel of the Simplon. The main heading was more of a center heading than a bottom heading, and, by working within it, the men had plenty of time to do their drilling in the face, which previously had always limited the progress. In this case it did not; the blasting was done as rapidly as the shovel could take the muck. That shovel, of about 40 tons capacity, was worked just as hard as any shovel in the open. It loaded full-sized, 12-yd. cars, and there was continuous and very good service rendered to the shovel, with no interruption.

The speaker believes this progress could not have been made if the rock had not been so exceptionally good, because timbering would have delayed the work, and probably the method would not have been successful if much timbering had been required. Eventually, it will be necessary to concrete more of this tunnel than has yet been done, because the rock may weather.

Another feature, not brought out very strongly in the paper, which contributed very largely to the success of this tunnel, was the drills used. The speaker believes they were little water-fed drills. With these it was possible to maintain the high rate of progress in the heading, and they contributed very largely to the success of the work.

The camp was very well kept, but the speaker was surprised to note that the power plant was of such small capacity. The working organization was very effective, and, without intermission, sustained the undoubted record-breaking progress with which this tunnel must be credited. This organization, the speaker is informed, was brought from the United States, where the men had done some very rapid work on the Gunnison and Laramie-Poudre Tunnels.

The wooden water pipe, used for ventilation, served its purpose well, and was much superior to the ordinary iron pipe. It was well laid, and the joints were so tight that the exhaust could be conducted through it.

T. Kennard Thomson,* M. Am. Soc. C. E.—Mr. Dennis deserves the thanks of the Society for his very interesting paper, but it is to be hoped that he will add many details before its final publication. Mr. Thomson.

* New York City.

Mr. Thomson.

The speaker understands that the report of the late Virgil G. Bogue, M. Am. Soc. C. E., is a masterpiece of one of the foremost of old-time railroad engineers, and it would seem that its publication in full, in connection with this paper, would be of great value to other engineers and students. For that reason it is hoped that the author will prevail on the Canadian Pacific Railway Company to allow its publication in full.

Mr. Dennis' paper is of special interest to the speaker because he was engaged on the location and construction of the Canadian Pacific Railway from Medicine Hat to the west slope of the Selkirks in 1883-84-85. Haste, the key-note then, as it seems to be still, resulted once in a record for rapid construction, when 93 miles of railroad from Medicine Hat was built in 23 working days of June, 1883. This work, which included grading, bridges, culverts, and track-laying, but not very much ballasting, really made a marvelous record, considering the fact that all the supplies had to pass through Winnipeg and then be carried some 600 miles over a new, single-track railroad.

In those days no one ever thought that that railroad would ever be a financial success; it was considered merely as a political necessity, as the Dominion Government, at the time of the Confederation in 1867, had assumed the obligation of making physical connections between the separated provinces. Even the late Commodore Vanderbilt had so little confidence in it that he got rid of his holdings.

As a matter of fact, in those days, British Columbia was closer, commercially, to the United States than it was to the rest of the Dominion; and, even in 1885, the speaker found that Ontario or Quebec banknotes were not welcomed in British Columbia and were accepted only at a discount, while United States notes were accepted at par. This was partly due to the fact that in those days the face value of a Canadian banknote was not guaranteed by the Government as it is now. At present, however, the Canadian banking system is superior in many respects to that in the United States where a national bank in a small town has only the credit of its small capital back of it, whereas the smallest branch of some dozen strong Canadian banks has its powerful system behind it. Very few Canadian towns are not amply provided with such branches.

One reason for haste in those days was to get some return on the money as soon as possible, and an even more important reason was the fear that, if the road was not quickly completed, a turn of the political wheel might prevent it from being finished for many years.

In addition to the commercial reasons for haste in constructing the Rogers Pass Tunnel, the fear of snow slides may have been another incentive—the author has stated that the snowfall there is from 30 to 50 ft. a year. The speaker assisted in the design of the

first snowsheds on this road. These were to be constructed in cuts on the hillsides in such a way that the snow slides or avalanches would never strike directly on the roof of the shed, but merely shoot or slide over it. Some of these designs were modified by others, so that the roofs projected up too far, with disastrous results in every case. Trees adjacent to a snow slide would often be cut off as if by a knife about 20 ft. above the ground, or at the surface of the standing snow, by the force of the wind generated by the falling avalanche. Nothing can be built to stop those giant snow slides after they have fallen from 3 000 to 5 000 ft. The snow and ice piled up in winter many feet, and even after the intense heat of August a depth of some 30 ft. or more would be left. Mr. Thomson.

Would it not be appropriate for the author to insert a brief sketch of the life of Major Rogers, one of the most unique characters of a past age of remarkable engineers? He saved the Canadian Pacific Railway from following the Columbia River some 200 miles around—north and then south—by discovering the Rogers Pass which led due west, crossing the Columbia River twice.

The exceedingly favorable geological conditions encountered in this tunnel were not often found in the earlier tunnels in the Selkirks, one of which—a mud tunnel—was nearly completed when it was found that the two headings did not meet by 18 in. This was first blamed on the engineer, but after the lining had been ripped out and replaced, the tunnel at once collapsed. It was re-opened a third time, and again collapsed, after which it was abandoned, and the railway was run around the bluff on a 23° curve. It was operated on this curve for more than 20 years, and then a new tunnel was built much farther from the face of the bluff, where firmer ground was encountered.

In those days some of the grades were as high as 4½%, and negotiating them gave the engines so much trouble that safety switches were generally placed at the foot of the grade, so that if the train ran away it would have to take the switch, which had such a sharp grade uphill that the momentum of the train would rapidly be reduced.

The plan and profile, Fig. 1, shows how the original location wound around the hillsides in order to avoid more excessive grades, and permitting as rapid and cheap construction as possible. The profile also shows why vertical shafts could not be considered, as the hills stand as high as 5 000 ft. above the tunnel.

Francis Lee Stuart,* M. Am. Soc. C. E.—The speaker has read this paper with interest, as the rate of progress was strikingly rapid. As far as the speaker knows, a pioneer tunnel, similar to that used at Rogers Pass, has never been built in the East, although this method is in constant use in coal and iron mines. Usually, the tunnels in Mr. Stuart.

* New York City.

Mr. Stuart.

the East have not been so long, have had less cover, and construction progress has been expedited by shafts. The speaker has found it very difficult to compare the progress in tunnels—every one seems to have different conditions which affect the rate of building—in fact, even parts of the same tunnel are not comparable. In groping after a greater speed, a number of new methods have been tried.

In the Stuart Tunnel, one of the most recent built under the speaker's direction, at the Magnolia Cut-Off, on the Baltimore and Ohio Railroad, the tunnel section used was large, being 31 ft. in the clear horizontally, with side-walls plumb to 11 ft. above sub-grade, and a semicircular arch with 15½ ft. radius, giving a clear height of 22 ft. 11¼ in. at the center of each track.

To expedite matters, the contractor, H. S. Kerbaugh, Incorporated, put in a Marion 28 (⅝-yd.), dipper, air-operated shovel with which he widened the heading, which originally had an area of about 120 sq. ft., and took the bench to an elevation 6 ft. below that of the wall-plate, leaving 11 ft. in bench instead of the usual 17 ft. It effected a very considerable saving in time and expense, as it was possible to widen and place the steel segment at a rate of from 60 to 70 lin. ft. of arch section per week, as compared approximately with 45 ft. by hand, and to enable the remainder of the bench to be drilled without cat-holes. The speaker thinks that matters were about even, as to cost, and, with a little better roof, it should have been cheaper than the old method.

In a further effort to increase the rate of progress in the tunnel, it was decided to use Blaw steel segments to support the roof, in place of the usual timber segments. These consisted of two 7-in., 14¾-lb. channels, riveted together with two ¾-in. rivets, 2½ in. long, with 6¾-in. fillers between, at intervals of about 3½ ft. These segments were divided into three parts; the central portion was 18 ft. 3⅞ in. long, curved to a radius of 17 ft. 6 in.; the end portions were 11 ft. 3⅝ in. long, 5 ft. 0⅛ in. being curved to a radius of 17 ft. 6 in., and the remainder being a tangent to connect with a flanged foot resting on the wall-plate.

The three sections were connected with 9 by ¾-in. plates, 1 ft. 9 in. long, and with ¾-in. fillers. They were used in 2 015 ft. of this tunnel, or for 60% of its length, and, for the most part, were 5 ft. from center to center; otherwise they were 7 ft. 6 in. from center to center, depending on the character of the roof. All were tied together longitudinally with ¾-in. tie-rods 7 ft. from center to center. The lagging was placed directly on these segments, and was fastened through $\frac{11}{16}$-in. holes in the flanges. The sections were assembled in the tunnel and raised to position.

The use of these steel segments enabled the driving to be done more rapidly, as the cross-sections decreased an average of about 2 cu. yd. per lin. ft., and, as the segments came entirely within the section of

the concrete lining, there was an additional saving of practically 2 cu. yd. of concrete per cu. ft. Mr. Stuart.

The speaker's conclusion was that speed was made by the use of these forms—whether or not they were cheaper could not be determined.

J. V. DAVIES,* M. AM. SOC. C. E.—The only thing the speaker has against this paper is its great brevity. The author has described a piece of tunnel work which has two peculiar and important features: first, the rapidity of the driving; and, second, the use of a "pioneer" heading. Mr. Davies.

To have done the amount of driving in the headings which was accomplished on this work for one month, is one thing, but to continue that work at the rates actually obtained, an average speed of 24 ft. a day for 7 days a week in one heading, and for 20 ft. a day for 7 days a week in the other heading, is really a wonderfully fine piece of work.

It indicates very clearly an extremely perfect organization in every department, and the speaker presumes that it is to be understood, from Mr. Lauchli's discussion, that that progress was in part due to the fact that the conditions, as they developed in the driving, and the conduct of the work, were almost perfect. There were no high temperatures; there was no entrance of hot water, as is so common in mountain ranges, and, in fact, very little water at all.

The comparison of that work with the best that has been obtained in the United States and elsewhere is interesting. The only tunnel in America (for which the speaker has figures) which exceeds the Rogers Pass monthly record (and that probably was only for an exceptional month), was the Red Rock Tunnel, built in 1901, in which a speed of 1·061 ft. for the month was attained; but that was in soft rock, where the boring was done with hand-augers.

The other tunnel to which the speaker refers is the Elizabeth Tunnel, on the Los Angeles Aqueduct, where Leyner drills were used in gneiss rock, and there the maximum progress was 641 ft. a month.

In the Roosevelt Tunnel, in Colorado, in granite, the speed was only 435 ft. a month. There, also, Leyner drills were used.

In an extensive undertaking of tunnel construction which the speaker's firm carried out in Mexico, a few years ago, heading progress was obtained, in hard limestone rock, as high as 536 ft. a month, using Leyner drills. The use of these drills has made a very marked increase in the rapidity of tunnel work.

Now, take the foreign tunnels: in the Simplon, in rock of a character probably very similar to this—hard, gneiss rock—a speed of 685 ft. was attained; in the Loetschberg Tunnel a rate was attained, in the north heading, in a hard limestone, of 1 013 ft., whereas in the south tunnel the rate was only 545 ft.; so that this Rogers Pass Tunnel work stands out pre-eminently as the most rapid that has been accom-

* New York City.

Mr. Davies.

plished in America up to the present day, and is almost equal to anything that has been done elsewhere.

There are various questions which arise from reading this paper, regarding which it would be of considerable value to the membership to have further information, and the speaker hopes that the author may feel willing to go outside of the scope of the paper as presented and give some further information which would add materially to its value.

The particular point of great interest is the value of the "pioneer" tunnel. The author refers to it briefly in the paper as an economical question, and it would be of much value if he could add something in the nature of a balance sheet, having on one side the advantages obtained from the construction of the additional heading, and on the other side the increased cost of driving two heading excavations, as was actually done.

The excuse and reason for a "pioneer" tunnel are unquestionably to increase the speed of construction of the main tunnel, in which there is a large bench to take out, as is involved in the enlargement of such a structure as this double-track railroad tunnel, where the value of the time element, in removing the bench and enlarging to full size, is the main factor involved on the one side of the balance sheet.

There is no doubt that speed can be attained by the driving of a "pioneer" tunnel, and undoubtedly in this case, as the author states, there was warrant for its construction.

Several of those discussing the paper have made remarks as to how the "pioneer" tunnel originated, but it seems to the speaker that any one who has studied the construction of the Simplon Tunnel, as described very fully by Sir Francis Fox,* would be satisfied that the use of the gallery in that work gave the idea from which the "pioneer" tunnel was adopted in this work.

It was used in that work unquestionably for the purpose of ventilation, for transportation and drainage, and to expedite the work.

On the other hand, there was another guiding reason in the Simplon for the adoption of the gallery, as the plans for that tunnel contemplated two single-track railroad tunnels, approximately 55 ft. apart, to be built for east- and west-bound traffic, respectively, and the "pioneer" tunnel was located as the side-wall heading for the second tunnel. It is now being enlarged, or has been in the last few years, to make the second tunnel; but it was located in that position unquestionably for the same purposes as the "pioneer" heading in the Rogers Pass Tunnel was driven, having in mind, undoubtedly, later enlargement for the second tunnel.

* "The Simplon Tunnel", *Minutes of Proceedings*, Inst. C. E., Vol. CLXVIII (1906–07), p. 61.

In the Simplon Tunnel the gallery was lined with the permanent side-wall construction at the same time. Mr. Davies.

In the Rogers Pass Tunnel, as described, there were none of the difficulties which were encountered in the Simplon Tunnel and made the use of a "pioneer" tunnel there so vital to the success of the undertaking.

In the Simplon Tunnel, the "pioneer" tunnel (or gallery, as it was called) was carried right through from end to end, and the gallery was enlarged so as to make a passing siding for trains in the center of the tunnel.

The speaker will be quite interested to obtain from the author at a later date some information as to what pressure he used in the ventilating system, with the Connersville blowers, under the conditions developed in driving this long-distance tunnel. The difference between any short tunnel and one of this length, with no intermediate shafts and no intermediate access, is one of the features of this undertaking which is of very great importance. The ventilation question becomes of the utmost importance, and the pressures at which the ventilating apparatus was operated to produce the forced draft for which the Connersville blowers were used, is of considerable interest.

Another matter on which the speaker thinks some further information would be of considerable benefit to the Society is a statement as to the detailed plan of the bonus system which was adopted for the acceleration of labor, as only a vague reference is made to the subject in the paper. The speaker would also be glad to have a schedule of the rates of wages paid to the men on that work.

S. A. Knowles,* Esq.—In reference to the speed at the Rogers Pass Tunnel, Mr. McIlwee, the original contractor, who was responsible for the methods and organization, had had similar contracts at the Roosevelt Drainage Tunnel, in Cripple Creek, the Laramie-Poudre Tunnel, and several others, in which he had been successful in maintaining a high average footage. The organization which was assigned at the Rogers Pass Tunnel had been with him on his previous undertakings, and thus their combined experience enabled him to obtain such desirable results. Mr. Knowles.

An essential factor in obtaining a high and uniform footage was the ventilation. The speaker had an opportunity at one time to test the ventilating system, which comprised 12 000 ft. of practically straight 18-in. steel pipe of No. 16 gauge. A No. 7 Root blower, running at 100 rev. per min., gave a displacement of 6 500 cu. ft., and the indicated horse-power required for either blowing in or exhausting this quantity of air per minute was about 20.

* Sault Ste. Marie, Mich.

Mr. Dougherty.

R. E. Dougherty,* M. Am. Soc. C. E.—The method used in driving this tunnel, which distinguishes it from the methods utilized in similar instances, involves primarily the so-called "pioneer tunnel." The arrangement is one which is somewhat new in tunnel driving, but the fact that the necessary results were secured by the contractors and the railroad company speaks for its success.

In sending out invitations to the contractors, the Chief Engineer of the Canadian Pacific Railway called attention to the fact that time was a very considerable factor, to such an extent as to be worth about $750 per day, and those who have had anything to do with railroad operation can very readily appreciate the significance of the time element. It is altogether possible that the tunnel might have been constructed by the so-called American methods, at a lesser cost, but the fact that all American records were broken, and the maximum results obtained by European methods approximated, would justify the means used.

Furthermore, from the standpoint of economy, reference might be made to an article by Mr. Sullivan, Chief Engineer of the Canadian Pacific, which appeared in one of the engineering periodicals, in which he states that the actual cost of construction was approximately $5.00 per cu. yd. of the main heading, from portal to portal, which included the cost of the pioneer tunnel, 5 miles of railroad to the site of the work, overhead charges, and all other costs involved. Furthermore, other bids secured by the railroad company ranged from $8.00 to $11.25 per cu. yd., although an estimate of $5.50 per cu. yd. was given by one contractor, based on the American method, but calling for a much greater length of time than was actually consumed. As a basis for comparison in the matter of progress, the maximum rate per month per heading attained at the Rogers Pass Tunnel was 946 lin. ft.; this approximates very closely the maximum rate of 1 013 lin. ft. per month attained in the construction of the Loetschberg Tunnel, in Switzerland.

In making a comparison of the methods used in the construction of other long tunnels, the only work which appears to have been on a similar basis is that involving the Simplon Tunnel in the Alps between Italy and France. The Simplon Tunnel was constructed for a single track, with a smaller auxiliary tunnel which was designed to become a part of a future parallel single-track tunnel, the prime purpose of the auxiliary tunnel having been to permit adequate drainage, as well as to facilitate the handling of material, etc. Without this auxiliary tunnel, it would have been almost impossible, in all probability, to construct certain portions of the Simplon Tunnel. In the case of the Rogers Pass Tunnel, however, the pioneer or

* New York City.

auxiliary tunnel had no relation whatever to drainage, inasmuch as apparently ideal conditions were encountered. Furthermore, the position of the pioneer tunnel, with respect to the main heading, was such as to render it of comparatively small value for drainage purposes, had unfavorable conditions of that character been encountered. Mr. Dougherty.

A careful study of the paper and other literature appearing from time to time in engineering periodicals certainly justifies the conclusion that, although the method used was not, in all probability, the cheapest that could have been adopted, nevertheless, the results were unquestionably consistent with the combined elements of economy and time.

One feature to which attention is attracted is the fact that only 1½ miles of the tunnel will be lined. It would seem that this may be a possible source of trouble in later years, especially as experience in the eastern section of the country indicates that, in a number of instances, tunnels unlined at the time of construction have had to be widened and lined over traffic at an abnormal expense; and, even under the most favorable conditions, it would seem that experience justifies the excavation of a tunnel section of sufficient area to permit at least of later lining without the necessity of excavating over traffic.

Congratulations are heartily extended to the author for his contribution to the literature of the Society, and primarily for the part which he has taken in the work under discussion.

C. R. HULSART,* ASSOC. M. AM. SOC. C. E.—Milton H. Freeman, Assoc. M. Am. Soc. C. E., Resident Engineer on the East River Tunnels, joins the speaker in considering the pilot heading somewhat as a horizontal shaft. Of course, its length is greater than the average length of shafts in tunnels, but it is much cheaper per foot. To justify pilot headings on a purely economical basis would be rather difficult. They must be justified by their convenience and the facilities they afford for expediting the work. For instance, the Walkill Pressure Tunnel, one of the Catskill Aqueduct tunnels, more than 23 000 ft. long, had six shafts ranging from 330 to about 500 ft. deep. Of course, such a pilot heading would not be applicable to pressure aqueducts, but, taking that as an example, for those six shafts at a cost of $200 per ft. for the sinking and lining (not a bid price, but cost), such a drift could have been built for one-half the length of the tunnel, or, in other words, shafts about ⅜ or ½ mile apart could have been used, instead of ¾ mile apart. Mr. Hulsart.

Tunnel ventilation is an interesting subject. The wooden stave pipe gives a certain rigidity, which was lacking in the 12-in. galvanized-iron pipe used throughout the Catskill Aqueduct. It permits of the exhaust method, which most tunnel engineers generally favor. With the galvanized-iron pipe, the exhaust method is somewhat disastrous, because in exhausting, while blasting, the pipe frequently col-

* New York City.

Mr. Hulsart.

lapses. The speaker has seen several hundred feet of pipe collapse, and this the wooden stave pipe would not do. In ventilating the Newark Bay Tunnel on the Passaic Valley Sewer, R. H. Keays, Assoc. M. Am. Soc. C. E., used a 6-in. pipe, and blew air into the heading at a pressure of 7 lb. at the compressor, which gave excellent results. The ventilation was effective for more than 1 mile. The length of the tunnel was 11 000 ft., and one heading was considerably longer than 1 mile. The men were never overcome by smoke or fumes.

On the Catskill Aqueduct, when air had to be driven for some distance, with a fan which could create a pressure of perhaps only 7 oz., a relay method was sometimes adopted, with one or two fans in the tunnel for blowing in or exhausting.

The contractors for the Rogers Pass Tunnel made extensive use of compressed air, a plan which not many New York contractors would follow, to the same degree. Compressed air is a very expensive form of power, especially (as in this case) when compressed to 1 000 lb., for use in traction engines, trucks, and blowers. On the Walkill Pressure Tunnel, which was particularly well planned, some tests were made with air compressed to 100 lb. and used for pumping purposes. Considering the whole system, from the power that went into the motors operating the compressors, to the water pumped at the tunnels, the efficiency was found to be about 7½%, and that of electric pumps, in small units, was about 55%; in other words, electricity was from seven to eight times as efficient as air compressed to 100 lb.

To compress air to 1 000 lb. requires two and a half times as much power, and it is used in any ordinary motor pump, hoist, or traction engine at a pressure of about 100 or 150 lb., so that the efficiency of such air-driven machinery would be about one-twentieth of that of electrically-driven machinery.

The Rogers Pass high-pressure plant required 1 500 cu. ft. at each end of the tunnel, or 3 000 cu. ft. of free air raised to a pressure of 1 000 lb. This involved the consumption of about 3 tons of coal per hour, during the time the two compressors were running (and it is assumed that they ran most of the time). Only one-twentieth of this fuel would have been required if electric power had been used for traction, power pumps, and blowers.

The shovel plates mentioned reminded one of the tendency, on a great many jobs, where everything is being sacrificed for speed, to forget to put them down, or perhaps eliminate them altogether. On one of the aqueduct jobs, where shovel plates were tried, the same gang mucked up the same quantity of rock in 30% less time with these plates than without them.

The cost of making records in driving in relatively small tunnels is worth studying. In the Catskill Aqueduct tunnel—equiva-

lent to a single-track railway tunnel—records were sometimes rather expensive, as the progress in such a tunnel is limited to a great extent by the speed of mucking. If the speed is increased the muck is increased, more muckers are required in a smaller space, and the men are in one another's way. As an example of this, in one of the headings of the Walkill Pressure Tunnel, where the progress had been about 80 ft. per week, each man was handling about 2 cu. yd. of solid rock in 8 hours. The progress was increased to about 90 ft., but in order to do that, the number of muckers was increased, and each laborer handled about 1½ cu. yd. of solid rock in 8 hours. The progress was then increased to somewhat more than 100 ft., and each laborer handled a little more than 1 cu. yd. of solid rock in 8 hours. In view of these facts, another firm of contractors, carrying relatively small overhead cost, separated, as far as possible, the drilling and mucking operations. The firm lost sight of records to a large extent, and went in for economical tunnel driving. Three gangs of drillers and three gangs of muckers were used. While the muckers were at one heading, the drillers were at the other, and when they finished drilling and shooting one heading, they exchanged headings with the muckers. The mucking was very efficient. The speaker does not know how much rock each man handled, but, it was generally considered as a very economical job. Mr. Hulsart.

J. G. Sullivan,* M. Am. Soc. C. E. (by letter).—The object of this discussion is to correct some erroneous accounts and misstatements which have developed in previous discussions. The most flagrant of the latter was the statement, by Mr. S. A. Knowles, that the credit for the quick work done at Rogers Pass Tunnel was due to Mr. McIlwee, who was the original contractor and was responsible for the methods and organization. This is very far from the truth. The facts of the case are as follows: Mr. Sullivan.

During January, February, and March, 1913, while the engineers were working on plans for the building of this tunnel, the writer had two or three conferences with Mr. Dennis, whose office at that time was in Winnipeg, and, on March 13th, 1913, he reported his ideas on the subject to the management of the Company in the following terms:

"Referring to the progress that we hope to make in driving of Rogers Pass Tunnel. I advised you in my report of October 22d regarding the relative speeds of driving tunnels on the American continent compared with those that have been driven through the Alps. I have given the matter considerable study since, and have come to the conclusion that the European method of driving a small lower heading and stoping out the remainder of the tunnel would be too expensive on this side, on account of the difference in the cost of labor.

* Winnipeg, Man., Canada.

Mr. Sullivan.

"I have been thinking out and studying methods that would tend to expedite this work. I first thought of driving a heading in the center of the tunnel, about 9 ft. by 12 ft., and keeping this heading close to the bench, carrying the air pipes over the muck in front of the steam shovel and into the heading. I still believe that this method in rock that will stand, is better than an upper heading. Mr. A. C. Dennis, however, suggested driving a pioneer tunnel and taking out an upper heading through shafts into this tunnel, taking out the rest of the bench with steam shovels. I pointed out to him that this was impracticable for the reason that from an upper heading you cannot drill to the bottom of the tunnel, and therefore would have to clean up all the muck in the bench before you could put in a round of breast holes to break more rock. I have now made plans showing a combination of my ideas and Mr. Dennis', which I think is well worth studying. The plan is to drive a small working pioneer tunnel, 8 ft. by 8 ft. underneath the main tunnel. I am sending you this for your information and, further, if this method should be adopted, that Mr. A. C. Dennis may have the proper credit for first suggesting a pioneer tunnel."

This was about a year before the writer ever saw or ever heard of Mr. McIlwee. So much for the methods adopted in the construction of this tunnel.

As to the statement that Mr. McIlwee was the original contractor: Mr. McIlwee never had any contract with the Canadian Pacific Railway Company in connection with this work. In the fall of 1913, the general contractors sub-let a portion of the work of driving the pioneer headings and the center heading to Mr. McIlwee. There immediately appeared, in some Denver papers, a notice to the effect that Mr. McIlwee had a contract for a 5-mile tunnel in Canada. The following is a quotation from the writer's report to the Management as to what was meant by these notices:

"The understanding was that he would bring men from Colorado and drive the outside pioneer tunnel at so much per foot at a certain rate of speed. The contractors were to pay him a bonus on any excess over this specified rate of speed. Contractors to furnish the cars, air, steel, and all other plant, as I wired you before, the result being that these men were working at piece work. If they do not make the speed guaranteed, they can be put off the work and nobody will suffer, as the entire plant is owned and controlled by the contractor. A man without one cent but brains, can enter into an agreement of this kind, while it takes capital to enter into a contract to build a five-mile tunnel."

In other words, Mr. McIlwee, on this particular work, might be ranked with the grade of contractors known as stationmen on railway construction work.

In reference to the discussion by Mr. E. Lauchli: Mr. Lauchli, in attacking this method of doing work, quotes the writer as authority for prices bid, without giving the full quotation, and leaving an incorrect

impression. In the article, from which the writer thinks the quotation was taken,* the full paragraph reads as follows: Mr. Sullivan.

"I may add further that, in reply to my invitation of April 8, 1913, the contractors who are doing the work, having in mind the method which was later adopted and which was suggested by myself and one of their Engineers, bid $6.10 per cu. yd., with a time limit of 42 months from date of signing contract. Other responsible and supposedly expert tunnel contractors bid from $8 to $11.25 per cu. yd., with time limits varying from 42 to 48 months. I do not know what method these latter contractors proposed to employ, but I always presumed it would be some modification of the European method."

All contractors had the same information, and the majority of those who bid sent men on the ground where they could see the character of the rock, from unbroken rock exposure, for practically the entire distance of the tunnel, from inspection along the railway, and also over the bare rock cliffs. These cliffs extended for a height of practically 6 000 ft. above the tunnel, which explains how absurd it would be to attempt to make borings in order to get any further information than was apparent from the surface; therefore, the meaning of the phrase previously quoted is entirely different from that placed on it by Mr. Lauchli. It simply indicates that the parties having in mind this method of boring the tunnel had confidence enough in it to know that they could build the tunnel 30 or 40% cheaper than by methods previously used.

Mr. Lauchli further states that his figures of "the cost of driving this bore, with a bottom heading, would be $5 per cu. yd., exclusive of contractor's profit." If this was a fact, all the writer can say is, that Mr. Lauchli's clients must do very well if they can make the proverbial German 1% profit, that is, of course, providing they get any work.

Mr. Keays is under a misapprehension when he states that "no heading was driven in the main tunnel, using the pioneer heading to work from." All the heading in the main tunnel was driven from drifts cut into that tunnel from the pioneer, and all the material from this main heading was carried through the pioneer tunnel, either directly to the outside, or around the shovels to standard-gauge cars back of where the shovels were working in the main tunnel.

Regarding Mr. Thomson's discussion: This is very interesting, and the writer is heartily in accord with his idea of paying homage and doing honor to the remarkable engineers of the past age, and especially to Major Rogers, whom the writer had the pleasure of seeing at a meeting of the Society, in Milwaukee, Wis., some 29 years ago. In this particular detail of the location of the Canadian Pacific Railway, however, Mr. Thomson has a wrong idea. The distance around by the Columbia River, from Beavermouth to Revelstoke, is 163 miles,

* *Engineering News*, February 24th, 1916.

Mr. Sullivan.

96 miles longer than the line located by Major Rogers, and for the traffic carried in 1912 and 1913, when the question of double-tracking and revising the line was being studied, it was seriously proposed to abandon the Rogers Pass route and build by way of the Columbia River, as almost enough could be saved in the cost of operation on this longer line to pay the interest on the construction of the new line, and a great deal more than the difference in the cost of the two lines, had the Company been fortunate enough to build around there in the first place. Nevertheless, the Company is now thankful to Major Rogers, for, had it originally built the line around by way of the Columbia River, the writer is sure that it would never have been abandoned for the present line, which, with the tunnel and heavy business, is more economical, especially in time, than operating the longer distance by way of the Columbia River. The Company should be especially thankful to Mr. Sammy Sykes, the Locating Engineer, who was responsible for the "Loops" on the west slope of the Selkirk Mountains, inasmuch as the original location ran straight down the north bank of the Illicillewaet, where, to this day, signs of some abandoned construction work can be seen, but, in order to save money at the expense of distance, Mr. Sykes put in the "Loops." The result of introducing these "Loops" brought the constructed line into the valley of the Illicillewaet, very close to the West Portal of the Tunnel.

These, the writer thinks, are two instances which illustrate the fact that it is never very safe to be too sure in a criticism. Here are two cases which could be looked on as blunders, as the increased cost of operation, in the one case on account of the heavy grades and the large rise to be overcome and in the other case operating over the longer distance of line without improvement in grades, with a reasonable volume of traffic, would be greater than the interest on the money that was saved in construction, but which really turned out to be a benefit rather than otherwise.

Mr. Davies asks for information regarding the economics of using the pioneer tunnel. To quote further from the writer's report of March 13th, 1913, to the Management:

"This method, of course, is only applicable where the rock will stand without artificial support, at least during the time of construction. Where the material must be artificially supported, then the top heading is the surest, and I think, the best way. The progress of the work by this method, as I said before, depends only on the speed that the pioneer tunnel can be driven. If rock is self-supporting, I see no reason why from 20 to 25 ft. per day could not be made. Placing the cost of driving the small tunnel at $30 per ft., that is the only part of the work that would be rushed under high pressure, and the heading proper can be taken out at least $5 per ft. cheaper than if the work

must be done under pressure, then the bench containing 18 cu. yd. per ft. (neat section) can, on account of there being no interruptions to wait for drilling or cleaning up to put in breast holes or knocking down material in order to get pipes into the heading, at a low estimate, be taken out 75 cents per cu. yd. cheaper, or $13.50 per ft., which would make a saving in excavation of tunnel proper of $18.50 per ft., leaving $11.50 to be taken care of in interest saved account making greater speed. In my report to you of October 22d, 1912, I estimated an annual saving of about $226 000, but all my figures were very conservative, and I took into account only one or two of the larger factors of the extra expense. Mr. Bogue's more accurate figures show a saving of over $370 000. However, his estimate for fuel per h.p. hour was 40% higher than the figure I used, and the price of coal was 17% higher than the price I assumed. His price is more accurate than the one I used, but assuming, for the sake of being conservative, that the average between the two estimates would be approximately correct, that would mean, say $300 000 per year saving, to say nothing of the interest on the $3 000 000 or $4 000 0000 that will be invested in construction, from which we will not be receiving any benefit until the work is completed. Therefore, if this tunnel can be completed one year sooner by using this method, the saving thus made will a great deal more than save the $11.50 additional cost of the pioneer tunnel." Mr. Sullivan.

That these estimates were conservative has been proved by the results, which were better than the estimates. The pioneer tunnel, from the most careful studies of the information at hand, cost about $28 per ft. instead of $30, as estimated. There was a great deal larger difference than 75 cents per cu. yd. between the actual cost of enlarging the tunnel by this method and the estimated cost of enlarging without the use of the pioneer tunnel; and another item, not taken into account in this estimate, was the fact that the pioneer tunnel only had to be driven less than four-fifths of the total distance. This was an indeterminate factor at the time these estimates were made, and the writer purposely omitted referring to it, as it was desired to have the estimates of this untried method conservative in any reports made to the Management.

It has been stated by some that this method is not applicable where there are soft spots in the rock. If the soft rock encountered does not exceed 50% of the total, the writer is confident that this method would still prove more economical than any other which has yet been tried, for the reason that when the soft places are encountered, there is plenty of time to stope out the upper part of the arch and timber it, so that when the steam shovel arrives at those places, there will be no delay whatever, and, instead of having to stope out the entire section by hand, as is necessary in the under-heading method, only about half, or less, of the material in the section requires to be removed in this manner.

Mr. Sullivan.

One word more as to actual costs: To quote further from the report of March 13th, 1913:

"I figure that by this method the pioneer tunnel can be driven for about $30, the main heading for about $40, and that the bench can be taken out for about $54, making a total of $124. There will be incidentals; contractors' profit should not amount to over $20 per ft. Of course, this method of driving the tunnel, working so many drills at one time, will require a larger plant than if only one heading was driven, and that at a slower speed than we contemplate."

The expectations of the Railway Company have been more than realized, as is proved by the speed and the cost of the work. The cost of driving this tunnel through rock, including in this price the cost of driving 19 610 lin. ft. of pioneer tunnel, twelve cross-cuts, each about 40 ft. long, erecting the plant (including freight), the proportionate cost of building about 5 miles of temporary railway tracks, and other overhead charges, plus 10% on all expenditures, will amount to a little less than $5 per cu. yd. for excavation in the tunnel proper.

In conclusion, the writer wishes to say that, in Europe, where it is understood that drill runners get from 90 cents to $1.25 and laborers about 75 cents, the method followed in driving the Loetschberg Tunnel may be both economical and rapid, but all the evidence at hand goes to show that where one pays from $5 to $6 per day for drill runners and from $2.50 to $3.50 per day for laborers, the method adopted is by far the most economical.

Mr. Dennis.

A. C. Dennis,* M. Am. Soc. C. E. (by letter).—The writer is very grateful to those who have been interested enough to discuss his paper, and wishes to thank them.

In reply to Mr. Lauchli, in his criticism of the Railway Company for its lack of geological data for use of bidders, it would appear that, except for the vicinity of the portals, the Railway Company could not, by boring, or other methods, supply information beyond what was available to the contractors for their inspection of the almost unbroken rock exposure from Rogers Pass to Bear Creek. The kind of rock and its pitch are obvious from inspection along the railway, and from the bare rock cliffs above.

The writer is unable to agree that the bottom heading method of tunneling is "the only method known to-day, which insures absolute success, both under favorable and unfavorable conditions." In good ground, it is obviously an economic question, depending to a large extent on the cost of labor. For heavy ground, such as that near the portals of this tunnel, it is believed that the bottom heading system is unsuitable.

* Vancouver, B. C., Canada.

Mr. Dennis.

There was heavy ground near the center of the mountain, in this tunnel, which required timbering. The main heading, from wall-plate level up, was excavated to full section by hand, and timbered by segments from the wall-plate set on rock. When the enlargement and shovel operations reached this point, no delay or damage to the timbering or bench on which the wall-plate was set was caused by blasting or mucking the bench. The work on this section was similar to that of timbering a heading and taking out a bench afterward, except that the muck going out and the timber, pipe lines, etc., in this case, did not interfere with other operations, as would have been the case without a "pioneer" heading.

Mr. Lauchli's figures for bids on this work, and his costs for other tunnels, do not appear to support the bottom heading system as being the cheaper method, even where cheap labor is available.

Mr. Shailer is quite right that 750 boiler h.p. would appear to be inadequate. These boilers, of 750 nominal h.p., however, readily produced 1 000 h.p., by reason of the induced draft. The fuel used was slightly less than the quantity estimated. The fuel estimate in figuring the bid was based on 22 lb. of coal per 1 000 cu. ft. of air compressed to 100 lb. by the gauge.

The compression of air beyond the pressure at which it is to be used, in order to provide for pipe-line losses, is an economic question which was given due consideration. It was calculated that the 8-in. main used was the economic size, and that a larger pipe would not save by reducing frictional losses to compensate for additional first cost, less salvage value, plus labor of putting down and removing. The higher air-pressure requirements noted are the maximum, and only apply for a short period. For a permanent plant, larger pipe would have been justified.

It is doubtful whether modern drills will result in much increase in tunneling speed by the "top heading and bench" system, as suggested by Mr. Keays. The heading drilling may be faster, but the bench drilling and mucking is usually the limiting feature in progress, and hammer drills are inefficient in long down-hole drilling. The writer must disagree with Mr. Keays' statement that the "ventilation between the working face and portal is unimportant", and also that the pressure system is preferable to the exhaust. The exhaust method only requires the mechanical removal of the air from 100 ft. or so of the heading, but the pressure system requires such removal for the entire tunnel length, in order to obtain a clear tunnel. The pressure system tends to mix the fresh air with the smoke and gas, rather than to drive the smoke out bodily. Wooden pipe, in the Northwest at least, solves the ventilating pipe problem, both from an economic and mechanical standpoint.

Mr. Dennis.

Mr. Lavis' words of appreciation are very welcome. The writer regrets that detailed cost figures, which are the real test of any work, have to be withheld to conform with the contractors' policy in this respect, but will go so far as to say that, under like conditions, even if time carried no bonus, he would adopt the pioneer method, unless there was sufficient time to drive the heading, centrally located, entirely through first, and muck the enlargement from one end only, and that never again would he use a top heading in hard rock.

The writer regrets that the report referred to by Mr. Thomson, by the late Virgil G. Bogue, M. Am. Soc. C. E., is not his property, and he is unable to give the Society the pleasure and instruction which was his privilege in studying this masterpiece by his old friend and Chief. Mr. Thomson, as well as many others of the construction days of the Canadian Pacific Railway, will regret that Major Rogers' name, like others who helped in the great work, will soon be forgotten on that railroad. The Rogers Pass Station has been abandoned, and the name of the Rogers Pass Tunnel has been officially changed to Connaught Tunnel.

To Mr. Davis, the writer would say that the idea of a pioneer heading originated in the desire to get away from the congestion, smoke, general confusion, and interference of one operation with another, observed in tunnel driving, and to provide muck in large quantities for handling by shovel. His work in coal mines, with the air course run with the main entry, suggested the pioneer as a means to this desired end. The Connersville blowers were used exclusively for exhausting from the working faces, and would produce 7 lb. below free air pressure. This provided satisfactory ventilation through 12-in. pipe for 1½ miles, at which point a second blower was put in, working as a booster on the same pipe line.

The general wages paid were 40 cents per hour to drill runners, and 35 cents per hour to others. The bonus probably averaged 25% in addition to these rates.

To Mr. Knowles, the writer has to say that the contractor he mentions is the one referred to in the paper as having a sub-contract for labor and explosives in certain headings for a short time, but whose sub-contract was cancelled on account of unsatisfactory work. Mr. White also seems to have been misinformed in crediting this former sub-contractor with the speed made. He and all his organization left the work a year before Mr. White's visit, at which time only a short length of heading had been driven in the rock.

In regard to Mr. Dougherty's point, as to whether this tunnel could not have been built for less cost by other methods, it is difficult to say. The writer does not think so, but would be much interested in a com-

parison of costs of other tunnels with those quoted from Mr. Sullivan's article, due allowance being made for extra cost for long tunnels, and bearing in mind that this tunnel was not in "ideal" or "made-to-order" ground. The West Portal earth section was very difficult, and the rock section was hard to break, as illustrated by the average explosives used in the pioneer heading, that is, 15 lb. of 60% gelatine per cu. yd. Mr. Dennis.

Mr. Dougherty's point, that all excavation should be taken out during construction to allow for lining, is well taken. The extra excavation can be done during construction for probably 10% of the cost of enlarging under traffic. No tunnel is entirely safe until it is lined.

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Paper No. 1391

UNUSUAL COFFER-DAM FOR 1 000-FOOT PIER, NEW YORK CITY*

By Charles W. Staniford, M. Am. Soc. C. E.

With Discussion by Messrs. Frederic R. Harris, D. A. Watt, C. A. Wentworth, Thomas H. Wiggin, T. Kennard Thomson, Charles S. Boardman, William M. Black, and F. E. Cudworth.

Synopsis.

This paper is descriptive of the design and construction of a steel sheet-pile coffer-dam of unusual type. This dam, so far as known, is the highest on record.

In outline the coffer-dam is a cellular steel sheet-pile structure, designed for a pressure head, of soft mud and water, of about 65 ft., having the cellular spaces filled with earth and clay to form the water seal, and held against the external pressure by an embankment of rip-rap or broken rock. All the steel sheet-piling forming the cellular spaces or "pockets" is in single pieces, many of which are more than 70 ft. long.

Descriptive and Historical.

On the refusal of the Secretary of War to grant any further extension of the pierhead line in the Chelsea District, thereby preventing permanent additions to the present piers and thus making it impossible to dock the new large transatlantic liners—more than 900 ft. long—at that locality, the City of New York was obliged to secure another site

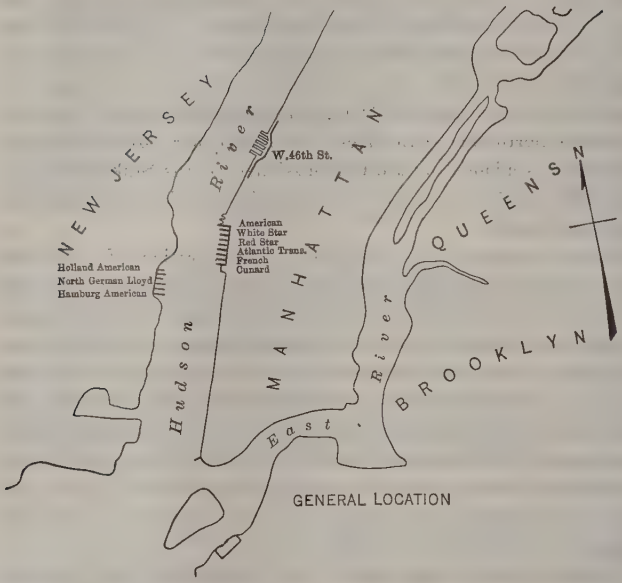
* Presented at the meeting of March 7th, 1917.

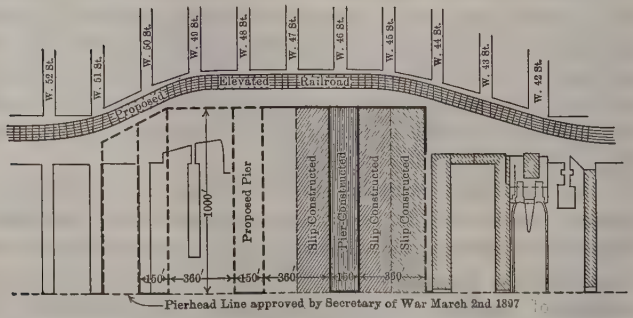
which would permit the construction of piers long enough for such vessels; and on April 10th, 1913, the Commissioner of Docks, Mr. R. A. C. Smith, presented a plan providing for a pier 150 ft. wide, at the foot of West 46th Street, extending 1 000 ft. inshore from the established pierhead line, and a half-pier at the foot of West 44th Street, with a slip room of 360 ft. between them, and a temporary slip, on the north side of the West 46th Street pier, 237 ft. 6 in. wide. This plan was adopted by the Commissioners of the Sinking Fund on April 30th, 1913, and steps were immediately taken to acquire the necessary property.

It was apparent from the general configuration of Manhattan Island in this vicinity that rock would be encountered over the inshore portion of the site of the proposed pier and slips. This rock surface was developed inshore from the existing bulkheads by wash-borings, and outshore from the bulkheads in the then existing slips by shod test piles. The development over the entire pier site out to the pierhead line, by these means and the usual test piles, indicated that the character of the river bottom varied from rock at an average depth of about 20 ft. below mean low water at the inshore end to a very soft mud at the outshore end, where it would be necessary to use fully lagged piles, from 85 to 89 ft. long.

In order to make proper depth of water for the giant liners, drawing, when loaded, about 38 ft., it was determined to provide for a depth in the slips of 44 ft. below mean low water, as records show that extreme low water has occurred 4 ft. below mean low water, and it was decided that there should be a clearance of at least 2 ft. between the ship's keel and the bottom of the slip, especially in that portion where rock forms the floor of the slip.

The sub-surface conditions thus developed and the decision to secure a depth of 44 ft. below mean low water in the slips plainly indicated the necessity for adopting a type of pier which would reduce to a minimum the quantity of submarine work to be done, and a method of procedure which would insure safe, workmanlike, and economical construction. Accordingly, a solid filled type was adopted for the inshore portion, about 220 ft. long, consisting of concrete retaining walls, to retain the filling and provide wharfage face, founded on bed-rock, thus utilizing the rock bottom over this portion of the pier area.





DETAILED LAYOUT

WEST 46th STREET IMPROVEMENT
FOR LONG STEAMSHIP PIERS

FIG. 1.

The larger quantity of rock excavation necessary with submarine blasting over that within the coffer-dam, in order to make certain that the excavation would be carried to the lines and grades required; the advantage of being able to control the surfacing of the rock for the pier and slip walls, and to prepare the rock surface for the foundation of these walls; and the desirability of building these walls "in air"; all indicated the wisdom of prosecuting the work within a coffer-dam to be built so as to enclose the entire site of the work, extending from the center line of West 44th Street northward to the center line of West 47th Street, a distance of 780 ft.

Design.

Having decided on the coffer-dam method of construction, the type to be used was made the subject of careful consideration. The area to be enclosed prohibited the use of bracing, and the unbraced steel sheet-pile type of coffer-dam was naturally suggested. Coffer-dams of this type had been used previously by the United States Government, at Black Rock and at Havana. There it was expected that the filling within the pockets would make them stable against the external pressure, unaided, but in both cases it was found necessary to place an embankment of stone against the pockets, and in the case at Havana, extensive bracing to the *Maine* was also placed, in order to stop the continuous inward movement of the coffer-dam.

In a coffer-dam of this type the internal frictional resistance of the filling takes the place of the cohesion in a solid, and analysis would show that, with material having a natural slope of not more than 3 to 1, failure would occur in vertical as well as horizontal shear, though the resultant passes through the middle third of the base.

The material overlying the rock at West 46th Street is a clayey mud, the natural slope of which is probably not as good as 3 to 1, thus prohibiting its use to obtain stability.

It had been decided to dredge over the site of the coffer-dam to 30 ft. below mean low water, leaving the remaining material to furnish support for the sheet-piles. The difficulty of removing the material from within the pockets and substituting better, as well as the greater cost of a coffer-dam of this type, due to the larger quantity of steel sheet-piling required, led to the adoption of the type shown by Fig. 2.

It should be noted that, where no embankment is placed against the pockets, the sheet-piles will be subjected to the pressure of the entire depth of filling at all times, and as the water after pumping may be expected to remain in these pockets for at least one-half their depth, if not more, as was the case at Havana, these sheet-piles will also be subjected to this hydrostatic pressure, necessitating a closer spacing of the diaphragms than where embankments are used.

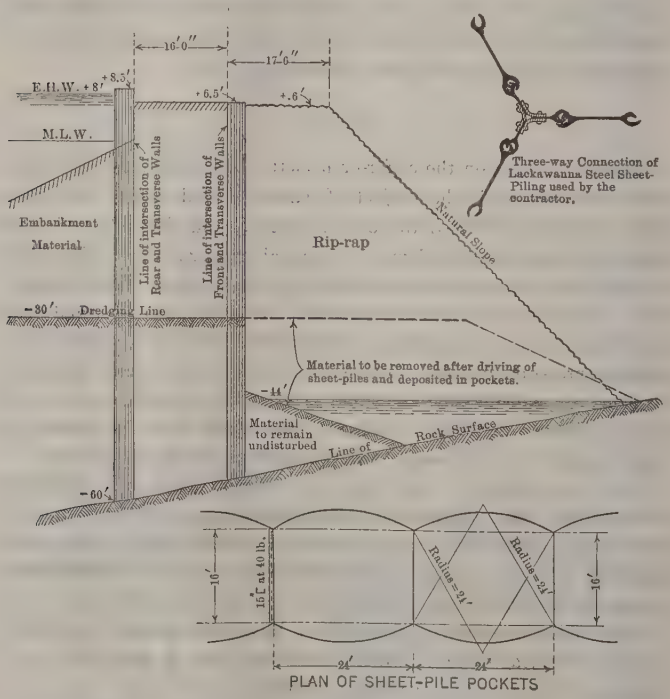


FIG. 2.

In the type of coffer-dam adopted, the sheet-pile pockets are used merely as a water seal, being supported against the external hydrostatic pressure by an embankment of rip-rap inside, and against the pressure of the rip-rap, during construction and before pumping, by a smaller embankment of earth outside.

This type of coffer-dam was adhered to, except along the crib bulkhead at West 47th Street, where, in place of the sheet-pile pockets, a single line of steel sheet-piling was driven, with an embankment of rip-rap in front, the space between the sheet-piles and the crib being filled with earth. Also, because of the limited space between the inner end of the West 44th Street Pier and the — 44-ft. rock contour line, two large cylinders filled with rip-raps were substituted for the pockets and embankment at this point. The only leaks, as described later, were encountered at these two places.

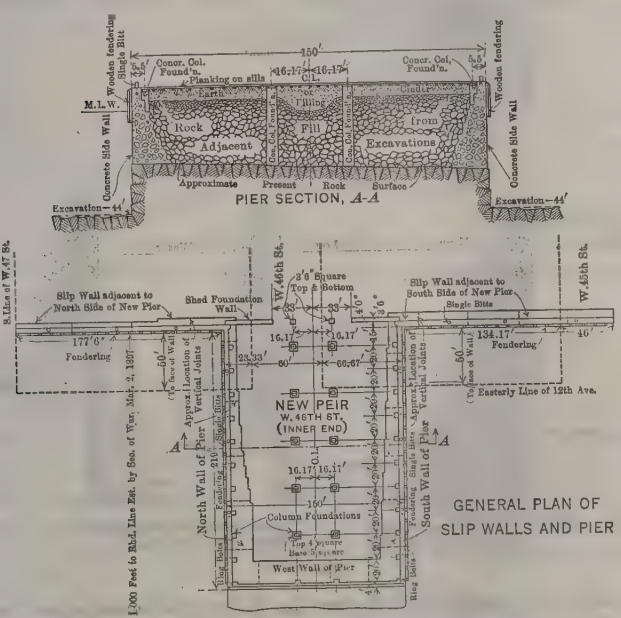


FIG. 3.

Pockets 6 and 22 were made 12 ft. wide and provided with sluiceways at a grade of 1 ft. below mean low water. These were put in chiefly to maintain the water at the same level on both sides of the coffer-dam during the construction of the closing pockets, thus avoiding any unbalanced pressure due to changes of tide before the placing

of the rip-rap embankment, and also to let the water into the coffer after the completion of the excavation and construction.

The specifications called for the following:

"The steel sheet-piling shall be rolled, interlocking sheet-piling similar and equal to that manufactured by the Carnegie Steel Company, type M-104, $\frac{3}{8}$-in. web, and weighing 38 lb. per lin. ft., or that manufactured by the Lackawanna Steel Company, $\frac{3}{8}$-in., straight web

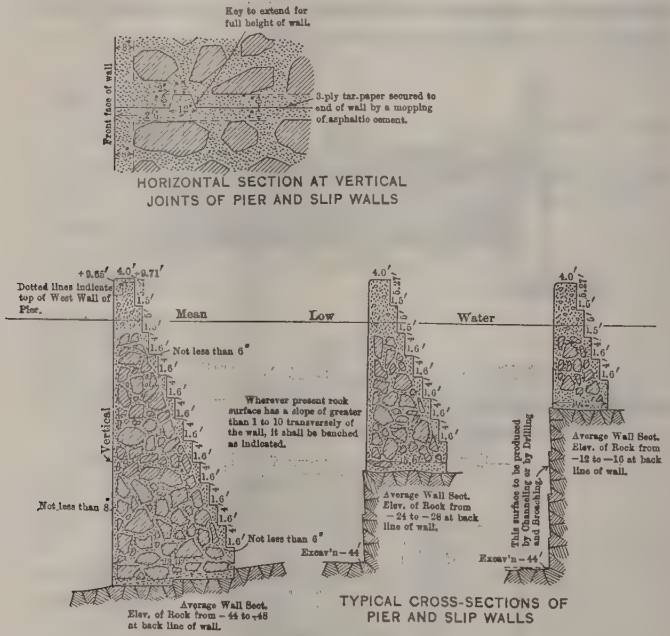

Key to extend for full height of wall.
Front face of wall
3-ply tar-paper secured to end of wall by a mopping of asphaltic cement.
HORIZONTAL SECTION AT VERTICAL JOINTS OF PIER AND SLIP WALLS
Dotted lines indicate top of West Wall of Pier.
Mean
Low
Water
Not less than 6"
Wherever present rock surface has a slope of greater than 1 to 10 transversely of the wall, it shall be benched as indicated.
Vertical
Not less than 8"
Not less than 6"
Excav'n — 44'
Average Wall Sect. Elev. of Rock from — 44 to — 48 at back line of wall.
Average Wall Sect. Elev. of Rock from — 24 to — 26 at back line of wall.
This surface to be produced by Channeling or by Drilling and Breaking.
Average Wall Sect. Elev. of Rock from — 12 to — 16 at back line of wall.
TYPICAL CROSS-SECTIONS OF PIER AND SLIP WALLS

FIG. 4.

section, and weighing 37.2 lb. per lin. ft., or that manufactured by the Jones and Laughlin Steel Company, 12-in., special I-beam section, and 5-in. locking bar, weighing, together, 36.1 lb. per lin. ft., and shall develop a tensional strength in the interlock between two adjacent piles, when tested on pieces about 3 in. long, of not less than 9 500 lb. per lin. in."

The contractor selected the type made by the Lackawanna Steel Company, shown on Fig. 2, which proved very satisfactory. The

three-way connection pile used at the intersection of the transverse and outer walls of sheet-piling weighed about 85 lb. per ft.

Stresses in Sheet-Pile Walls of Pockets.

Due to the pressure of the retained filling, the sheet-piles are all in tension transversely. Their resistance to this tension is limited by the strength of the interlock, which tests show will exceed 9 000 lb. per lin. in. When the filling is done in a proper manner, the pressures from each side of the transverse walls will balance each other, and the tension will be entirely due to the pressure against the longitudinal sheet-pile walls.

$$t = \frac{p\,d}{12};$$

where t = the tension, in pounds per linear inch, in the transverse walls;
p = the pressure, in pounds per square foot, against the longitudinal walls at the level considered; and
d = the distance between the transverse walls, in feet.

The longitudinal sheet-pile walls are arcs of a circle, and, assuming them to be subjected to a radial pressure from the filling, the tension per linear inch is:

$$t_1 = \frac{p\,R}{12},$$

where R is the radius, and $t_1 = t$ where $R = d$.

The original plan required the pockets to be filled with the material remaining on their inshore sides after dredging. It was considered possible, therefore, that they might be entirely filled before depositing the rip-rap embankment. In this case the sheet-piles would be subjected to the pressure of about 58 ft. of filling. Assuming the weight of the submerged river mud at 80 lb. per cu. ft. and its natural slope at 3½ to 1,

$$p = 0.57 \times 80 \times 58 = 2\,600 \text{ lb. per sq. ft.},$$

and as $r = d$, the maximum stresses in the longitudinal and transverse walls are equal and given by

$$t = \frac{2\,600 \times 24}{12} = 5\,200 \text{ lb. per lin. in.}$$

The possibility of avoiding unduly straining the sheet-piles by depositing some of the embankment before completely filling the

pockets was considered, and was actually found advisable, as explained later.

Tests on Model Embankment.

To obtain information regarding the probable behavior of a rip-rap embankment under various conditions of pressure, some tests were made with a model, one-twentieth the size of the embankment, placed against the inshore side of the sheet-pile pockets, or 42 in. in height

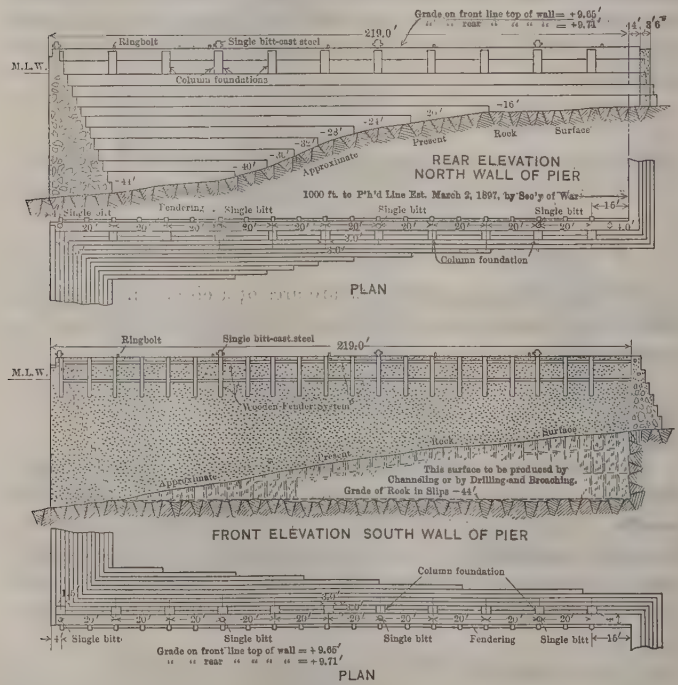


Fig. 5.

at the rear. These tests are incomplete, but, though it was found necessary to discontinue them, they are given here for whatever value may be derived from them.

The testing apparatus provided a bottom having the average slope of the rock floor at West 46th Street, 6 to 1, and also glass sides, 3 ft. apart, in order to observe the effect of pressures on the mass. The

board, against which the embankment was placed and by which the pressures were transmitted to the embankment, was fitted so as to slide inward on the bottom and between the glass sides. The embankment consisted of broken stone weighing as placed about 93 lb. per cu. ft., and varying from $\frac{1}{4}$ to $\frac{3}{4}$ in., which is about one-twentieth the size of rip-rap. The pressures were obtained by using two calibrated steel car springs, which were compressed by a nut working on a threaded bolt, passing partly through each spring, and fastened to the backing blocks attached to the base of the apparatus. The pressures applied were determined from the decrease in length of the springs, a gauge being used for each spring, giving the pressure directly in pounds.

In order to determine the behavior of the embankment under pressure, vertical lines were marked on the ground-glass sides, and along them columns of small wooden blocks were laid up in the embankment. These columns of blocks moved with the embankment, and the extent of their displacement from their original positions was measured as the pressure was increased. Figs. 6, 7, 8, and 9 are reproduced from photographs showing various views of this model.

Six tests were made with this model under different conditions, with the results shown by Figs. 11, 12, and 13.

In Test 1, the board transmitting the pressure extended the full height of the embankment, or 3 ft. 6 in. The embankment had a berm of 10 in., and the pressure was applied at one-third the height from the bottom.

In all the tests the embankment was 3 ft. 6 in. high, but, in all but Test 1, the board was only 33 in. high, being placed against the upper 33 in. of the embankment, the lower edge of the board being approximately in the horizontal plane through the toe. The pressure in Tests 2, 3, and 4 was applied at one-third the height of the board from its lower edge. Tests 5 and 6 will be referred to later.

The object of these tests was to determine the extent of the distortion which would take place in such an embankment under pressure, as well as its ultimate resistance.

Figs. 11 and 12 show the distortion throughout the mass in Tests 1 and 2, obtained by measurements of the displacement of the blocks of wood referred to. Also, the curves of deflection of the embankment with the increase of pressure are shown by Fig. 13.

In Test 1, at a pressure of about 2 375 lb., a tendency to break, about through the horizontal plane through the toe, was observed, and is indicated by the breaks in the lines in Fig. 11 at this pressure, as well as in the break in the curve of deflection shown in Fig. 13.

The pressure above the horizontal plane through the toe corresponding with 2 375 lb. on the whole board is 1 460 lb. The weight of the embankment above this plane is about 1 900 lb., and the corresponding value of μ is 0.77, as compared with 0.83 indicated by the natural slope of 1.2 to 1.

A second break occurs in the deflection curve in Fig. 13 at a pressure of 2 750 lb. Assuming this as an indication of sliding up the inclined bottom, taking the weight of the embankment at 2 380 lb., the tangent of the angle made by the bottom with the horizontal at $\frac{1}{6}$, and substituting those values in the formula,

$$\mu = \frac{P - \text{tan. } \alpha\ W}{W + \text{tan. } \alpha\ P},$$

we have a value for μ, the coefficient of friction of the stone on the wood bottom, = 0.83. This high value is presumed to be due to the probability of the stones becoming embedded in the wood to some extent.

Considering the break in the deflection curve for Test 2, Fig. 13, at 1 280 lb. as an indication of sliding along the horizontal plane through the lower edge of the pressure board, the weight of the embankment above this plane being about 1 750 lb., the resulting value of μ for broken stone on broken stone is 0.73. The critical pressure of 1 000 lb. in Test 3 gives a value for μ of 0.80. These values compare with 0.83 indicated by the natural slope of 1.2 to 1.

The movement of the embankment in these tests, accompanying each increase of pressure, would apparently cease in a very short time. The readings of both pressures and movement were then taken, and the next increase of pressure effected. It became evident, however, that, particularly for the higher pressures, further movement will occur when more time is allowed. This is illustrated in Test 2, Fig. 12, under pressure 8, in which case the pressure against the embankment dropped from 1 805 to 1 565 lb. over night, accompanied by a corresponding movement of the embankment. To determine the effect of time, Test 4 was made on an embankment similar to that used in

Fig. 6.—Side View of Model, Showing Displacement of Columns of Wooden Blocks at End of Test 1, When the Pressure Against the Embankment had Reached 2 810 Lb. The Black Vertical Lines Show the Original Position of the Columns of Wooden Blocks.

Fig 7.—Front View of Model, Showing Slope of Broken Stone Embankment.

FIG. 8.—SIDE AND REAR OF MODEL, BEFORE APPLICATION OF PRESSURE IN TEST 2, SHOWING COLUMNS OF WOODEN BLOCKS LAID UP ALONG BLACK LINES ON GLASS SIDES.

FIG. 9.—SIDE VIEW OF MODEL AT END OF TEST 2, WHEN PRESSURE HAD REACHED 1 810 LB. NOTE DISPLACEMENT OF WOODEN BLOCKS.

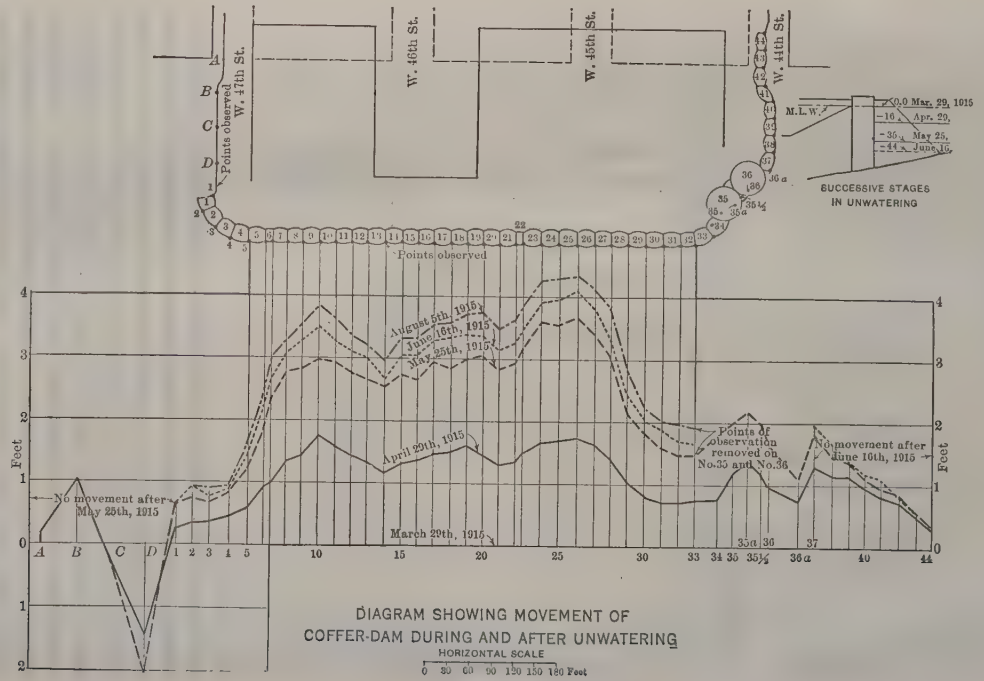


DIAGRAM SHOWING MOVEMENT OF
COFFER-DAM DURING AND AFTER UNWATERING

HORIZONTAL SCALE

0 30 60 90 120 150 180 Feet

FIG. 10.

Test 2, with a berm of 10 in. The springs were compressed until they indicated a pressure of 1 060 lb. The accompanying movement of the embankment at the top was 0.07 in. At the end of 1 week, the springs indicated a pressure of 950 lb., and this drop in pressure was accompanied by an additional movement of 0.045 in. at the top of the embankment. At the end of 1 month, the pressure had dropped to 900 lb., with a further displacement of the embankment at the top of 0.08 in. Observation of the embankment at the end of the second month indicated no further change either in pressure or displacement.

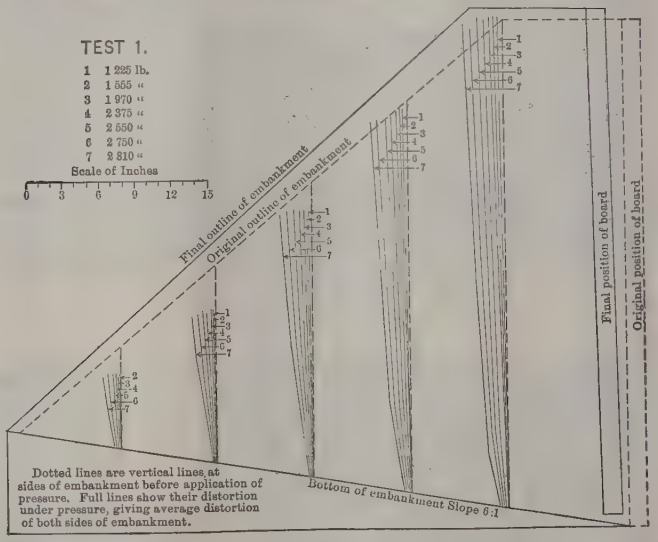

TEST 1.
1 1 225 lb.
2 1 555 "
3 1 970 "
4 2 375 "
5 2 550 "
6 2 750 "
7 2 810 "
Scale of Inches
0 3 6 9 12 15
Final outline of embankment
Original outline of embankment
Final position of board
Original position of board
Dotted lines are vertical lines at sides of embankment before application of pressure. Full lines show their distortion under pressure, giving average distortion of both sides of embankment.
Bottom of embankment Slope 6:1

FIG. 11.

This test does not show that the embankment would not have come to rest in time under a pressure greater than 900 lb. constantly applied, which could not be determined with the apparatus used, as any movement of the embankment necessarily results in a partial release of the springs, and consequently a decrease in pressure.

Tests 5 and 6 were undertaken to determine the effect of applying the pressure at a point above the center of gravity of the embankment as well as the effect of friction along the face of the embankment in contact with the board.

The embankment used in these tests was the same as that in Test 2, except that the width of the berm was about 6 in. In the case of Tests 5 and 6, the pressure was applied at two-thirds of the height above the bottom of the board. In Test 5 the board was free to move, as in the previous tests, and in Test 6 the board was anchored at the bottom with hinges. The results are given by the curves of deflection of the top and bottom of the board, as shown in Fig. 13.

The curve for Test 5 indicates an ultimate resistance of about 600 lb. to a pressure applied at two-thirds of the height, which about

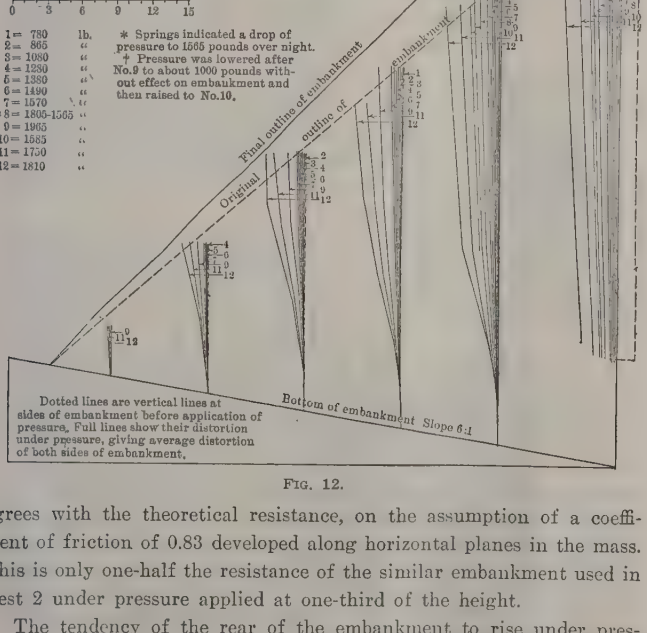
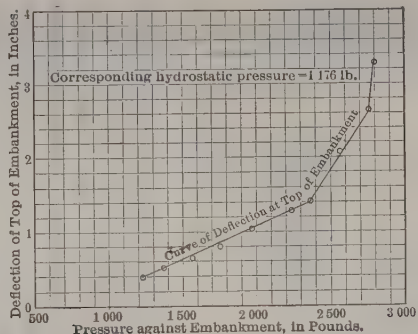

TEST 2.
Scale of Inches
0 3 6 9 12 15
1 = 780 lb.
2 = 865 "
3 = 1080 "
4 = 1230 "
5 = 1380 "
6 = 1490 "
7 = 1570 "
* 8 = 1805-1565 "
† 9 = 1965 "
10 = 1585 "
11 = 1750 "
12 = 1810 "
* Springs indicated a drop of pressure to 1565 pounds over night.
† Pressure was lowered after No. 9 to about 1000 pounds without effect on embankment and then raised to No. 10.
Final outline of embankment
Original outline of embankment
Dotted lines are vertical lines at sides of embankment before application of pressure. Full lines show their distortion under pressure, giving average distortion of both sides of embankment.
Bottom of embankment Slope 6:1

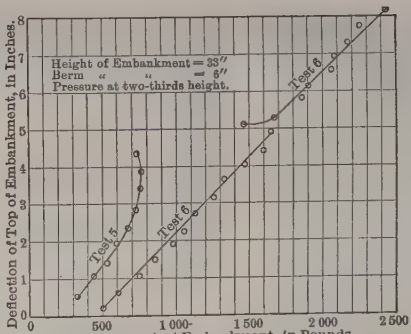
FIG. 12.

agrees with the theoretical resistance, on the assumption of a coefficient of friction of 0.83 developed along horizontal planes in the mass. This is only one-half the resistance of the similar embankment used in Test 2 under pressure applied at one-third of the height.

The tendency of the rear of the embankment to rise under pressure, carrying the board with it, was very evident in Tests 1, 2, and 3. In Tests 5 and 6 the movement was entirely horizontal. To determine the effect of friction between the board and the face of the embankment, the board, in Test 6, was anchored as mentioned pre-

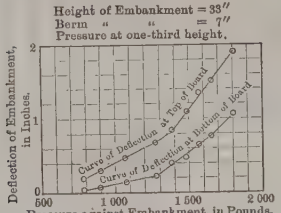

Corresponding hydrostatic pressure = 1 176 lb.
Curve of Deflection at Top of Embankment.
Deflection of Top of Embankment, in Inches.
Pressure against Embankment, in Pounds.

Test 1.

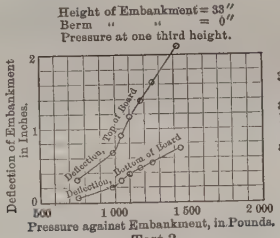

Height of Embankment = 33″
Berm " " = 6″
Pressure at two-thirds height.
Test 5
Test 6
Test 6
Deflection of Top of Embankment, in Inches.
Pressure against Embankment, in Pounds.

Note:— In Test 6, the pressure board was fastened at the bottom by hinges.

The break in the curve is explained by a considerable slide of stone down the slope of the embankment at the time.


Height of Embankment = 33″
Berm " " = 7″
Pressure at one-third height.
Curve of Deflection at Top of Board
Curve of Deflection at Bottom of Board
Deflection of Embankment, in Inches.
Pressure against Embankment, in Pounds.

Test 2.


Height of Embankment = 33″
Berm " " = 0″
Pressure at one third height.
Deflection at Top of Board
Deflection at Bottom of Board
Deflection of Embankment in Inches.
Pressure against Embankment, in Pounds.

Test 3.

FIG. 13.

The embankment was in all cases 3 ft. long.

The hydrostatic pressure for a height of 33 in. is 730 lb.

The height of the embankment is given as measured from the bottom of the pressure board.

The slope of the embankment was in all cases about 1.2 horizontal to 1 vertical.

viously. The result is shown by the lower curve, Fig. 13. Considering the resistance developed along horizontal planes in the embankment, and assuming a value of 0.83 for the coefficient of friction for both stone on stone and stone on wood, the ultimate resistance in this case should be about 2 500 lb. Under the maximum pressure developed in this test (2 400 lb.), no indication of failure was evident. The excessive deflection of the embankment made it impracticable to carry this test further. The friction along the face in contact with the board would evidently be more effective the lower the point of application of the pressure. These results should be of value in the design of bulkhead walls.

The theoretical active pressure for the embankment in Test 1 would be about 280 lb., and that in Test 2, about 170 lb. If the curve for Test 1, Fig. 13, is extended, it will indicate a zero deflection at 800 lb., and for Test 2, a zero deflection at about 600 lb. In Test 4, the board rested against a wale near the top before the pressure was applied, a thin slab of wood having been placed between the board and this wale. Any movement of the embankment would release the slab of wood, and this occurred when the springs indicated a pressure of 850 lb., checking the results obtained from the deflection curve as closely as could be expected. These results seem to indicate a maximum resistance, without deflection, of about three times the active pressure.

Construction.

Pile-Driving.—The driving of the sheet-pile pockets in the designed shape was accomplished by driving four timber piles in the location for each pocket, and these served to support a plank templet slightly smaller than the size of the pocket, around which the steel piles were set. The piles of the first five pockets were set in such order that the closure occurred in the inshore panel, the remaining pockets being set so that the closure came in the diaphragm common to adjoining pockets, it being easier to get a good fit for the closure pile in the straight diaphragm than in the curved inner panel.

In setting the piles, their own weight caused them to settle in the mud to a depth of about 20 ft., so that they were required to be driven only about 10 ft. The driving was done with a "New Monarch" type of steam hammer, weighing about 4 000 lb., bolted to a frame which fitted over the piles and held the hammer in a vertical position, the

hammer plate being wide enough to drive three piles at once. After being driven in sets of three, the individual piles were driven with a follower, in order to insure contact of each with the rock. As the first five pockets were in the area formerly occupied by the inner end of the West 47th Street pier, a considerable quantity of timber was encountered, which made the setting and driving of the piles difficult. In the case of Pockets 3, 4, and 5, timber was encountered near the surface of the mud, which caused some of the piles to project so high when they were set that it was necessary to drive them with a steam hammer down to grade, which would allow the adjoining piles to be entered. This resulted in throwing Pockets 4 and 5 so far out of plumb that the piles adjoining the closure pile in each pocket were farther apart at the bottom than at the top. The hard driving necessary to draw these adjoining piles together caused the rupture of one jaw of the closure pile in Pocket 5, leaving a gap in the inner panel of about 25 ft. from the rock bottom. A similar condition was produced in placing the closure pile in the diaphragm between Pockets 8 and 9, it being impossible to detect the fact that the closure pile had passed out of the adjoining jaws.

There was no difficulty in driving the piles to rock between Pockets 8 and 35, the bottom having been dredged to 30 ft. below low water, removing practically all the timber usually found embedded in the upper surface of the mud.

At the beginning of the return at West 44th Street, however, a great deal of trouble was caused by the proximity of the two large cylindrical pockets to the inner end of the 44th Street pier. Before dredging was done at this point, a timber crib occupied a large part of the area covered by Pockets 35 and 36, extending under the inshore end of the pier shed. In dredging with a dipper-dredge it was impossible to clear out the bottom of the crib close to the shed on account of the danger of damaging the structure, consequently some of the piles in Pockets 36 and 35½ encountered the projecting ends of the round crib timbers. Very hard driving was required to put down the piles which were close to the crib, four piles in Pocket 35½ requiring 24 hours of driving with a 2-ton steam hammer. Apparently, these four piles passed through three courses of crib logs, as the very bad driving occurred in three stages, yet there were no indications that they were parting from each other or curling up, and they finally brought

up solidly on the rock. Evidently, however, there was a rupture, as practically all the leakage of the entire dam took place through the outshore side of Pocket 35½.

The setting of pockets, which was begun with No. 1, on April 10th, 1914, was continued until No. 27 was reached, on June 19th, 1914. Owing to a change in plan, setting was then suspended until October 5th, when it was resumed in No. 35, the cylindrical pocket, and carried thence to Pocket 44. Pockets 28 to 30 were then set, Nos. 31 to 33 being omitted until the lighter had completed the removal, from the rear of the dam, of all the mud which it could not reach from a position outside the dam. The lighter was then placed outside the dam and the remaining pockets were set, the closure being made in Pocket 31 in the middle of the inshore and outshore panels.

Dredging and Filling Pockets.—The weakness at these two points was soon developed when the filling of the pockets was begun. In the case of Pocket 5, the mud had been dredged from the inshore side to a depth of about 30 ft. below low water, the material being placed in the pocket to a grade of about 7 ft. above low water. There was thus a head of about 37 ft. of mud on the inner panel, which, after the lapse of a few hours, completed the opening of the gap at the closure pile.

In order to repair the damage it was necessary to remove some of the filling which had been placed in Pockets 6 and 7, and thus allow the corner pile between Pockets 5 and 6 to be pulled back into a vertical position. The distorted piles of the ruptured panel were removed and new piles were placed, two being added to the original number to fill the space caused by the spreading of the corners when the filling was suddenly released.

The failure of the diaphragm between Pockets 8 and 9 occurred when the filling in No. 8 had reached — 12, and in No. 9 when it had reached — 20, and the mud inshore from the two pockets had been dredged to about — 40 ft. The original plan, in constructing the dam, was to dredge the material in the rear overlying the rock, place it in the pockets, and then proceed to deposit rip-rap at the rear; and, when the rip-rap had reached the level of the mud, outshore from the dam, to deposit filling outshore in sufficient quantity to counteract the pressure of the rip-rap.

The failure of Pockets 5 and 8 made it evident that this procedure would have to be modified, unless there was assurance that the closure piles were not under considerable initial stress. A number of the closure piles had required considerable driving, and it was decided to avoid unduly stressing them by proceeding as follows: The mud in the rear of the dam was excavated for the length of two pockets and taken to sea; rip-rap was then deposited until it reached the grade of the original bottom in the pockets; filling was then placed in the pockets, and a head of mud and earth of about 10 ft. over the grade of the rip-rap was maintained as the placing of rip-rap was continued. Sufficient tension was thus maintained in the walls of the pocket to make them water-tight without subjecting them to severe loads. The filling comprised a mixture of mud and cellar earth, the latter being dumped from trucks which were driven along a platform built on top of the dam. As the earth was dumped in the middle of the pockets, its greater weight forced the mud up along the walls, providing a seal which proved to be absolutely water-tight.

During the time required for the foregoing operation of dredging and filling, which averaged about 10 days for each pocket, the unbalanced pressure of the mud in the pockets and also that outshore from the latter caused them to lean inshore. The mud in which the piles were set is of a stiff, clay-like consistency which will stand at a steep slope for a considerable time if undisturbed. The impact of the 2-yd. buckets of mud on the original filling in the pockets apparently was sufficient to destroy the cohesion in the mass, causing it to exert pressure tending to tip the pocket inshore. The maximum movement of a pocket (No. 16) immediately after the completion of the dredging was $\frac{1}{2}$ ft. in 24 hours, and there was a maximum total movement of about $2\frac{1}{2}$ ft. in the 10 days required to restore the balance of pressure on the sides of the pocket. The average movement during the time of placing the filling was about $1\frac{1}{2}$ ft. The balance of pressure, once restored, was easily maintained by placing the earth embankment outshore to counteract the pressure of the rip-rap.

In dredging and filling the cylindrical pockets (Nos. 35 and 36) a different procedure was necessary, on account of the change in the type of construction. Whereas, in the small pockets, the pressure of the mud inside tended to hold them in the same shape as when they were set, in these cylinders, as it was necessary to clear the inside

FIG. 14.—LAYING OUT TEMPLATES FOR DRIVING SHEET-PILING.

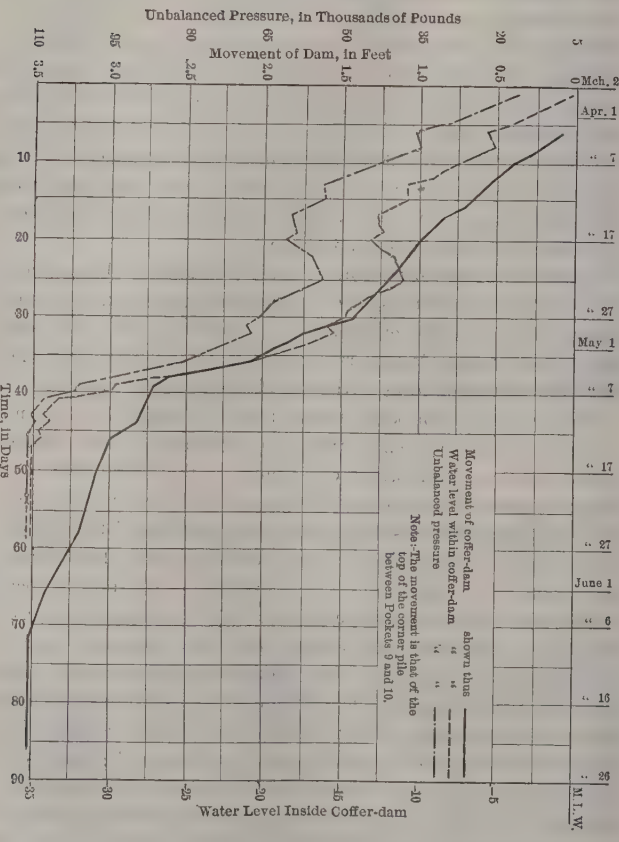
FIG. 15.—SHEET-PILING FOR POCKETS DRIVEN.

Fig. 16.—Driving Steel Sheet-Piling for Coffer-Dam.

Fig. 17.—Large Pocket, and Rip-Rap Embankment, West 44th Street.

of mud in order to bring the rip-rap in contact with the rock bottom, there was some danger that the pressure of the mud around the outside might distort or collapse the cylinders. This danger was avoided by dredging around the outside of the cylinders at the same time that deepening was going on within, except on the sides which were adjacent to the crib. Here the proximity of the crib precluded any dredging, and it was necessary, in order to hold the pockets in shape, to leave a fillet of stiff mud around that portion of the circumference which received pressure from the outside. In each cylinder the rock was cleared of mud for about three-quarters of the area of the bottom. Immediately after the completion of dredging the rip-rap was deposited uniformly around the circumference, maintaining practically the circular shape throughout the operation.

Pumping.—The unwatering of the coffer-dam was begun on March 29th, 1915, with a Morris 10-in. centrifugal steam pump, set on skids resting on the rip-rap embankment. Steam for this pump was furnished by a 50-h.p. locomotive boiler placed on the top of the dam. With this pump the level of the water was lowered from 0 to — 13 ft. in 19 days, the pump being operated only 16 hours per day, in order to allow the rip-rap embankment to consolidate itself. Meanwhile (April 13th), a leak began developing in the single line of piling along the face of the crib at 47th Street, about 100 ft. from the river, and, by April 18th, this had reached the capacity of the pump. The single line of piling which forms the dam along 47th Street was driven on a line parallel to, and from 4 to 5 ft. from, a rock-filled timber crib which extends along 47th Street. Along the face of this crib, at the point where the leakage took place, there was none of the stiff mud which acted as a seal elsewhere along the dam. Furthermore, the rock fill in the cells of the crib was so porous that the river water had free access to the piling, and, although earth filling was deposited between the face of the crib and the sheet-piling, its thickness (about 4 ft.) was not sufficient to prevent a considerable flow between the piles and the rock. In order to place against the foot of the piling material which would not be drawn through by the flow of water, wooden piles were driven at intervals of from 4 to 5 ft. between the steel piling and the face of the crib. These were then withdrawn, and the holes filled with a mixture of sand and horse manure by ramming down charges of this material with the pile until the stiffness


Unbalanced Pressure, in Thousands of Pounds
110 105 100 95 90 85 80 75 70 65 60 55 50 45 40 35 30 25 20 15 10 5
Movement of Dam, in Feet
3.5 3.0 2.5 2.0 1.5 1.0 0.5 0
Mch. 28
Apr. 1
" 7
" 17
" 27
May 1
" 7
" 17
" 27
June 1
" 6
" 16
" 26
M. L. W.
10 20 30 40 50 60 70 80 90
Time, in Days
35 30 25 20 15 10 5
Water Level Inside Coffer-dam
Movement of coffer-dam shown thus
Water level within coffer-dam " "
Unbalanced pressure " "
Note: The movement is that of the top of the corner pile between Pockets 9 and 10.

MOVEMENT OF COFFER-DAM FROM BEGINNING OF PUMPING.

Fig. 18.

of the driving indicated that the cavity was filled. By thus introducing material which would clog easily, and by consolidating the earth filling for about 50 ft. along the sheet-piling, the flow was reduced to small proportions.

During the 9 days required to stop this leak, the water level rose from — 13 to — 11 ft., although the 10-in. pump had been running for 24 hours per day for the last 4 days. On April 22d the level again began to recede, the pumping plant being supplemented on April 29th by two 6-in. and one 4-in. Alberger centrifugal pumps, electrically operated. These were on a raft within the coffer-dam. After setting up the electrically operated pumps they were run continuously, and the 10-in. steam pump was run for 16 hours per day. In 16 days (April 22d to May 8th) the water level was reduced from — 11 to — 34 ft., at which point the accumulated mud in the bottom of the area was exposed.

At about this time water began to flow through Pocket 35½, appearing at the bottom of the piles in the inshore panel. It was then discovered that the closure piles of this inshore panel had parted from the adjoining piles for several feet from the bottom, probably due to driving the panel in a curve of too small a radius. It appeared that the leak occurred in the outshore panel of Pocket 35½, where there was a possible rupture due to driving through the timber crib. The same method as used at 47th Street—of driving down manure to hold the earth filling—was applied along the outside of Pocket 35½, and, although the flow was reduced, it was impossible to cut it off entirely. Coincident with the removal of the mud in the bottom of the excavation, the water level was lowered until the final grade of — 44 ft. was reached. The three small pumps were removed, and a 12-in. Worthington centrifugal pump, electrically operated, was placed on the same platform as the 10-in. steam pump, on the rip-rap embankment at a point about 30 ft. below low water. The 12-in. pump was thenceforth used to keep the dam dry, the 10-in. pump being maintained as a reserve. The total inflow amounted to about 2 500 gal. per min., of which probably 2 000 gal. came through Pocket 35½, and the remainder from the cut-off along 47th Street and from the upland. There was no indication of any leakage whatever through the portion of the dam composed of the small cells, and it is evident that, had conditions permitted their use throughout, the leakage would have been slight.

During the progress of unwatering, daily observations of the movement of the dam were made, so that, if necessary, the rate of pumping might be changed in order to check undue movements, but the changes were at no time so great as to cause apprehension. The maximum inshore movement at any one point in 24 hours was 0.40 ft. at Pocket 25, when the water level was lowered from — 24 to — 27 ft.; and the total movement during the 40 days required to remove the water to — 34 ft. was 2½ ft. The dam continued to lean inshore after pumping had been completed to — 44 ft., with a slowly decreasing rate, coming practically to rest by June 15th, 1915. The total inshore movement of the top was about 3½ ft.

Fig. 10 shows the alignment of the coffer-dam for several stages of the unwatering, and Fig. 18 shows the history of the movement of a typical section of the coffer-dam as the unwatering proceeded. The unbalanced pressures were calculated on the assumption of a hydrostatic head from rock at the inner row of sheet-piles at — 60 to + 4.

Plant, and Rock Removal.

The plant provided for handling the excavated material and concrete comprised eight 15-ton steam locomotive cranes, of which six were used in the slip and pier areas and two on the old 45th Street pier. These narrow-gauge steam locomotives provided transport for concrete and spoil. The six cranes used in the coffer-dam were placed so that all parts of the area could be covered, three being carried on timber trestles extending inshore from the dam, the other three partly on trestles and partly on the two pier side-walls and the slip wall at 44th Street. There were two lines of narrow-gauge track along the top of the dam extending out on the old 45th Street pier, where the two cranes loaded the rock into scows lying alongside the pier. Spurs extended to the bulkhead along 47th Street and to the cranes on each trestle. There were also three stiff-leg derricks with 70-ft. booms, one at 47th Street, about 50 ft. outshore from the bulkhead wall, one at the north side of the pier, in the rear of the bulkhead wall, and one near 44th Street, at the bulkhead. These derricks were operated by compressed air, supplied by the plant provided for the rock drilling.

In placing the trestles for the locomotive cranes, a crane was set up on top of the dam, and the mud at the toe of the rip-rap embankment was excavated with a 1½-yd. "battleship" which operated some-

FIG. 19.—LEVEL OF WATER INSIDE COFFER-DAM, MAY 1ST, 1915, 17.9 FT.

FIG. 20.—CONCRETE CORNERS AND LOWER COURSE FOR RETAINING WALL, SEPTEMBER 10TH, 1915.

what after the manner of a clam-shell bucket. As the toe of the slope was beyond the reach of the crane boom, several 24-in. I-beams were laid on the rip-rap, and the bucket was allowed to slide down them. After the rock had been uncovered, a trestle bent was erected at the foot of the bank, and an intermediate bent half-way up the slope. These bents were spanned by five 24-in. I-beams on which the crane track was laid. Successive bents were erected as the rock excavation proceeded, the trestles finally being extended so that the cranes would reach the bulkhead.

The rock excavation was begun at the toe of the rip-rap bank, and carried thence toward the bulkhead. Ingersoll-Rand tripod air drills were used until the depth of the cut was about 22 ft., and then Cyclone well-drilling machines, operated by gasoline engines, were substituted. The small drill holes, which were started with a 3½-in. and finished with a 1½-in. bit, were about 4 ft. from center to center. The well drill holes were 6 in. in diameter, and were about 14 ft. from center to center. Along the line of the pier and bulkhead walls, line drilling was done with the tripod drills, the holes being from 6 to 8 in. apart.

There was considerable difficulty with some of the small drill work, on account of the seamy nature of a part of the rock; this was avoided by the use of the well drills. The rock in this locality is composed of hard gray granite and Manhattan schist, with little order or arrangement in their occurrence. In the outshore portion of the excavation, the rock proved to be a fine, sound granite, which furnished a good foundation for the pier side-walls, but, in the area east of a line about 75 ft. from the bulkhead, granite and schist were encountered in irregular layers. The strata of the schist stood nearly on edge, as is usual, and varied from a hard to a quite soft, much laminated, structure. The granite, particularly within the area of the bulkhead wall, was badly broken up by seams, running in all directions and mostly filled with mud. In a number of locations the rock was of such poor character that it was necessary to excavate within the wall area, down to the grade of — 44 ft., in order to provide proper support for the front of the wall.

The rock was disposed of by loading it by hand into steel "battleships" and wooden skips, which were hoisted by the locomotive cranes to flat cars of about 3 cu. yd. capacity. These cars were transported

by locomotive to the pier at West 45th Street in trains of from six to eight cars, and there unloaded by two locomotive cranes into bottom-dumper and deck scows. The rock thus loaded was shipped to Riverside Park, between West 88th and West 95th Streets, and there deposited along the shore in a trench, so as to form an embankment for the extension of Riverside Park. Other portions of the rock excavated were used for back-filling in the rear of the pier and bulkhead walls, and for the cyclopean concrete.

Pier and Slip Walls.

The plant for the concrete work was erected on a pile platform, about 30 ft. wide and 100 ft. long, built adjoining the outshore side of the dam near the 47th Street end. On the platform there was a cement shed with a capacity of 4 000 bags. A 1-yd. Ransome mixer, electrically operated, was placed at an elevation which allowed it to discharge into buckets set on flat cars on the track running along the top of the dam. Hoppers for stone and sand, of 90 and 45 cu. yd. capacity, respectively, were erected directly over the mixer, resting partly on the dam and partly on the platform. A timber stiff-leg derrick, with a 70-ft. boom, operated by compressed air, and set at the same elevation as the hoppers, supplied them with sand and stone from scows moored alongside, using a 1½-yd. clam-shell bucket. The cement was elevated to the charging hopper by a chain bag conveyor run by an electric motor. For use in cold weather, steam pipes were inserted in the sand and stone hoppers and in the mixing water tank. In operation, a flat car with two bottom-dumping buckets, each holding a batch, was run under the discharge chute of the mixer by a locomotive, whence it was transported to the locomotive cranes which placed the concrete in the forms.

In the construction of the slip walls, the rock on the foundation was thoroughly cleaned of all loose and rotten pieces, washed with a stream of water, and given a coat of mortar. The concrete was brought up in steps of 4 ft. and bonded together by allowing the derrick stone placed in it to project from one course into the other. Steel forms, in sections of 5 by 10 ft., were used, except for the face of the wall for the top 14 ft. Here a wooden form was used, in order to produce a better surface and to facilitate the placing of anchor bolts for the fender system. All the pier and bulkhead walls were built in sections 30 ft. long, each section being tarred to provide an expansion joint.

Fig. 21.—Drilling, Blasting, and Removing Rock, July 9th, 1915.

Fig. 22.—Within the Coffer-Dam, July 9th, 1915.

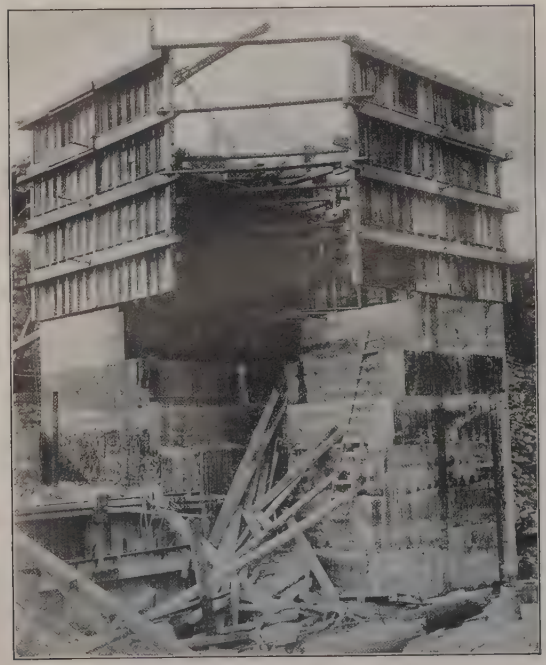


FIG. 23.—OUTSHORE CORNER OF CONCRETE PIER WALL, SEPTEMBER 14TH, 1915.

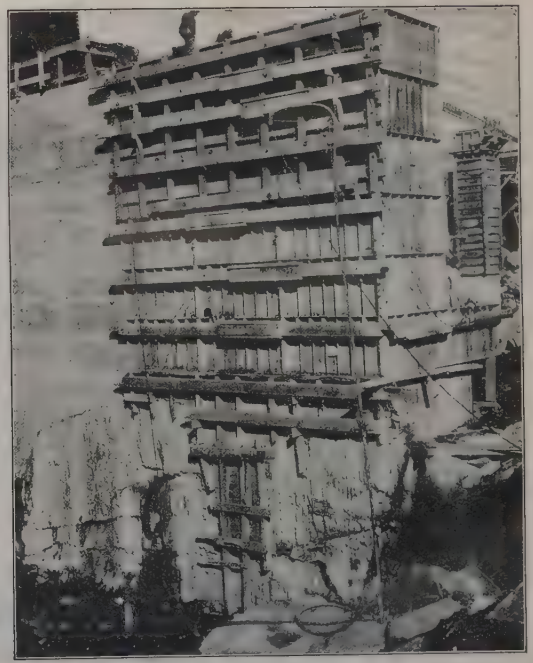


FIG. 24.—SIDE FACE OF SOUTH PIER WALL, SHOWING COMPLETED CONCRETE, LINE-DRILLED ROCK FACE, AND WALL FORMS ABOVE, NOVEMBER 6TH, 1915.

FIG. 25.—ROCK FACE AND CONCRETE RETAINING WALL, NORTH SIDE OF PIER, NOVEMBER 6TH, 1915.

Rewatering.

The rewatering of the coffer-dam was effected on June 29th, 1916, by opening the sluice-gates when the tide was at about the low-water stage, which gave a head of 1 ft. of water over the bottom of the sluiceways. The rising of the tide increased the head until, at the expiration of about 6 hours, it had reached about 5 ft., the average during the rise of the tide thus being 3 ft. The clear width of each sluiceway was 7 ft. 8 in. To fill the dam to its capacity of about 8 000 000 cu. ft., 8 hours were required. Practically no movement of the dam was caused by the rewatering.

Removal of the Coffer-Dam.

Preliminary to pulling the sheet-piling of the dam, the fill in the pockets and in the outshore embankment was removed to a depth of about 10 ft. below low water, in order to relieve some of the pressure, and jet pipes, operated by compressed air, were forced down along the piling nearly their full depth, in order to break the hold of the mud on the piles.

The plant for pulling the sheet-piling comprised a 25-ton and a 40-ton steam lighter for the main portion of the dam, and stiff-leg derricks for the returns extending inshore at the upper and lower ends of the dam, equipped with inverted McKiernan-Terry steam hammers operated by compressed air. The hammers were attached to the piles with 3, 3½, and 4-in. pins, inserted in a single hole burned in each pile about 2 ft. below the top. Various types of grips were also used, but the most successful was a relatively light one furnished by the Lackawanna Steel Company, which was rigged so that it could be opened by pulling a trailing rope when the pile had been pulled out of reach from the ground. This grip, which catches the pile at the top of the web, has been used continuously by one lighter for all but the hardest pulling.

The pulling was started where the piles were from 65 to 70 ft. long, and considerable difficulty was experienced in taking out the first pile, about a week being consumed in trying various piles before one was extracted from the diaphragm of Pocket 28. The pulling of the diaphragms and outshore panels of Pockets 18 to 29 was then accomplished by driving them up with the hammer.

It was found impossible, however, to start any of the piles next to the rip-rap embankment by these means. A 100-ton lighter was

secured with the intention of using it, with an inverted hammer, to break one of the inside arcs. After working 4 days and trying several pockets, this rig, being unable to budge any of these inside piles, was dispensed with. A heavy, guyed, A-frame was then erected on top of the rip-rap embankment and rigged with a set of eight-sheave blocks, with sixteen parts of ¾-in. line, through which the pull of the 25-ton derrick was transmitted to the pile. A 64-ft. pile was thus drawn by direct pull without the aid of the hammer. The line of piles adjacent to the rip-rap having thus been broken, no further difficulty was experienced in this direction, and the remaining piles were drawn by the derrick aided by the steam hammer.

As a rule, all the piles came out straight and in a condition satisfactory for use in other work.

After the removal of the sheet-piling, the filling, outshore earth embankment, and inshore rip-rap embankment will be removed by dredging.

Cost of Work.

Dredging.—The dredging was executed under two separate contracts, first for the original contract, and then over the area of the supplementary contract. Under the first contract, 271 882 cu. yd. were removed, at a cost of 23½ cents per cu. yd. Under the supplementary contract, 87 875 cu. yd. were removed, at a cost of 27 cents per cu. yd., making a total, for the removal of material by dredging down to solid rock, of $87 618.52.

Coffer-dam, Rock Excavation, Walls, etc.—After due advertisement of the specifications and plans, based on an engineer's estimate of $497 000, for the coffer-dam and walls of the pier, the Department obtained thirteen bids, ranging from $750 000 to $487 000, the low bid, which was awarded to Holbrook, Cabot and Rollins, Incorporated.

In order that the work might be extended southward to West 44th Street, to comprise an area which it was not possible to include at the time of the original preparation of the contract, a supplementary contract was entered into with the same contractors at the same unit prices, after receiving permission from the Board of Estimate and Apportionment and the Board of Aldermen to extend the work in this manner.

It was clearly demonstrated at that time that a saving of fully $150 000 would be made by doing the work in connection with the

original coffer-dam, thereby avoiding the cost of building a new coffer-dam at the south end of the work.

The total of the original and supplementary contracts for the work of building the coffer-dam, excavating the rock, and constructing the concrete walls for the inner part of the pier building, was $626 000.

The unit price for the removal of solid rock under the contract was only $1.60 per cu. yd., but assuming that one-half the cost of the coffer-dam should be charged against rock excavation and the other half against the construction of the concrete walls and other items within the coffer-dam, the actual cost for the removal of this solid rock, was about $5.31 per cu. yd.

Conclusion.

First.—Instead of excavating this large quantity of subaqueous rock by the "dry" method, the work might have been performed without a coffer-dam by the ordinary method of rock blasting under water, or, in other words, the excavation of the rock "wet".

If it were possible to perform this work so as to obtain under water vertical face walls to the exact limits of the slip, it would have cost, under ordinary normal prices for this class of work, four or five times as much as by the method adopted.

It is not possible to conceive, however, that the work could have been done under water, for, as a matter of fact, it was very difficult to do it with line drilling in the open, and further, even were it possible to deposit concrete in a satisfactory manner at such depths, part of the work in addition would have been very expensive.

The particular result obtained by the construction of this dam was that, although all preliminary examinations, borings, soundings, etc., were made when the whole territory was covered by filling of all kinds, including, in places, cribwork, coal yards covered with coal, buildings, etc., the total amount paid under the contract obtained through unit prices has been kept close to the original estimates.

Second.—The whole river face of this dam, for a length of about 800 ft., under its full water head, was absolutely dry, all the pumping, during the whole construction period having been done by one 12-in. centrifugal pump. Furthermore, all this pumping would have been avoided, although much more was expected, had it not been for the presence of cribwork at the north side, and on account of the impossi-

bility of driving two piles in the large cylinders at the south end, because, in driving, a boulder or some other interference was encountered, rendering it impossible to drive these two piles home for a distance of only about 1 ft.

Supervision.

The work was executed by the Department of Docks and Ferries, R. A. C. Smith, Commissioner of Docks, and R. C. Harrison, First Deputy Commissioner.

The design, preparation of plans and specifications, and the construction was under the direction of the writer, as Chief Engineer, assisted by R. T. Betts, M. Am. Soc. C. E., Deputy Chief Engineer; Mr. T. F. Keller and Elias Cahn, Assoc. M. Am. Soc. C. E., Assistant Engineers. The work in the field was under the direction of J. J. Pemoff, M. Am. Soc. C. E., Assistant Engineer.

The contracting firm for the work was Holbrook, Cabot and Rollins, Incorporated.

Acknowledgment is made by the writer of the skill and perseverance of the contractors, particularly of T. B. Bryson, Assoc. M. Am. Soc. C. E., on the work. The writer's thanks are also due to Brig.-Gen. William M. Black, M. Am. Soc. C. E., Chief of Engineers, U. S. A., for advice in the preparation of the plans, and the support of his assurance that this work could be accomplished successfully provided proper construction methods were used.

DISCUSSION

Mr. Harris.

FREDERIC R. HARRIS,* M. AM. SOC. C. E. (by letter).—The author has presented his subject so clearly and fully that extended comment is unnecessary.

On first reviewing the paper one is impressed particularly with the large sum of money involved for the construction of only 220 ft. of pier and the excavation of areas adjacent to the pier sufficient for three slips. However, when it is considered that it was necessary to limit the projection of the pier outshore, it will be realized that the sum expended for the inshore construction was not exorbitant. It must also be taken into account that the greater part of the steel sheet-piling was salvaged and was available, therefore, for use on other work.

In coffer-dam construction, as in other constructions of this class, the unknown element confronts the engineer. Officers of the Corps of Civil Engineers of the Navy have been forcibly impressed with the hazards accompanying construction of this class, as there is little room for doubt that most of the delays and failures in naval dry dock construction are traceable to weak or faulty coffer-dam construction. When both the design and construction of a coffer-dam are imposed entirely on the contractor, there is a strong temptation to save on first cost and perhaps to take chances on additional and unnecessary risk. The contractor is seldom to blame for this condition, as he is often forced into an untenable position by the double competition involved, first, in competitive bids in which his estimated cost of the coffer-dam which he plans is a part; and, second, in which the saving that he may believe he can make on the coffer-dam determines his profit, or, worse still, a decrease in his loss.

The writer has been much interested in the construction work described, having discussed it quite frequently with Mr. Staniford, Messrs. Holbrook and Bryson, of the Holbrook, Cabot and Rollins Company, and the Commissioner of Docks and Ferries, Mr. R. A. C. Smith, and has been influenced largely by Mr. Staniford's able and conservative engineering plans, as shown by this work, to depart from the precedent heretofore established in the plans and specifications for naval dry docks. This departure, although it has not taken the exact form of designing the coffer-dam required for the work, has, nevertheless, laid down and specified a minimum coffer-dam requirement that would be acceptable to this extent, attempting, as Mr. Staniford did in the work described, to minimize the risk and safeguard against the probability of failure, with its attending losses of money and time.

* Washington, D. C.

Mr. Harris. The fact that the coffer-dam described by the author held and served most satisfactorily the purpose for which it was constructed, is evidence of the soundness of the judgment used in its design.

The writer is particularly interested in the tests made to secure information on the behavior of the rip-rap embankment under various conditions of pressure. The following are the results of computations made from the data in the paper:

In Test No. 1, assuming the head to be 3 ft. 6 in., the active pressure was found to be 360 lb.; the pressure at impending motion, 1 455 lb.; and actual sliding on the bottom 2 800 lb., assuming the same coefficient of friction on the bottom as that of the material itself, and no friction on the board to which the force is applied. These assumptions may account for the figures given being slightly larger than those obtained in the test.

In Test No. 2, the head assumed was 2 ft. 9 in. The computed active pressure found was 250 lb.; the computed impending motion, 970 lb.; and for sliding, 1 460 lb., assuming a plane of rupture through the toe and no friction on the board to which the force is applied.

In Test No. 5, the head was assumed to be $1\frac{1}{2}$ times the distance of the application of the force from the top. At impending motion, the computed force was found to be 500 lb. and the sliding on the plane through the toe to be 680 lb.

From observations of the curves shown in Fig. 13, it is believed that the initial movement was probably purely local, and that the impending motion began at the initial points of the curves shown. The agreement of the computations with the critical points of the curves is remarkable, and partly verifies such an assumption.

Mr Watt. D. A. Watt,* M. Am. Soc. C. E. (by letter).—It would be of interest to know whether any piles were found, during the removal of the pockets, which had been driven out of their interlock with the next piles. In the writer's experience, this is by no means uncommon where the driving is hard, for instance, with closing piles, although no indication of the break may be apparent at the time. Thus, when removing the coffer-dam for the *Maine* at Havana, more than one case was found where one pile at one-quarter or one-third of its length from the end had broken away from its neighbor. Fortunately, the breaks were all below the level of the harbor bottom, which doubtless was the reason the cylinders had not burst, as the surrounding material (mud and clay) prevented the embedded piling from spreading. The piles had been driven from 30 to 35 ft. into this material. During the construction of the coffer-dam one of the cylinders broke open, due to the closing pile being driven out of the interlock, but in this

* Albany, N. Y.

case the weak point appeared to be above the harbor bottom, where there was no outside material to counteract the strain. Mr. Watt.

Similar conditions were found in the later steel-pile-pocket coffer-dam at Troy, N. Y., where the piles were driven into river gravel. The penetration was not more than 8 or 9 ft., as the gravel was compact; yet, when removing the piling, several cases were found where the interlock had driven out. As with the *Maine* coffer-dam, the breaks appeared to be below the surface of the bottom, and no harm resulted.

The saturation of the filling in coffer-dams of this type, referred to by the author as one of the sources of internal pressures, appears to exist at a higher level than is usually assumed, although varying naturally with the porosity of the filling. The filling used for the *Maine* cylinders varied from a sandy to a heavy clay, and in all cases the saturation line was very high. It began at about mean tide level on the outer faces and sloped downward at about 5 horizontal to 1 vertical. The material, therefore, was dry only to an average of about 5 ft. below mean tide; the remainder stayed in a fluid condition. Efforts to lower the saturation level and to drain the filling with long perforated pipes were useless; the pipes would fill up in a few hours after being pumped out, and even near the surface these did not seem to affect visibly the adjacent seepage. This high and constant line of saturation was noteworthy because the total seepage into the coffer-dam was extremely small (about $\frac{1}{2}$ cu. ft. per sec.), and there was as much opportunity for the water to leak out of the cylinders into the inclosure as to leak into them from the bay. In the Troy coffer-dam, with the pockets filled with sand and gravel, the water line was naturally much steeper. Judging from the height of the seepage appearing on the inside faces, it was about 2 horizontal to 1 vertical.

The writer visited the 46th Street coffer-dam during its construction, and again after it had been in use for some months. Mr. Staniford and the others responsible for the work are to be congratulated on their success in handling a very difficult problem.

C. A. Wentworth,* M. Am. Soc. C. E.—The Society has received a valuable contribution in this paper, describing an advanced step in coffer-dam construction to meet peculiar and difficult conditions. The tests made in preparing for this work show a foresight which is too often omitted. A comparatively small expenditure for borings and tests at the outset of a difficult piece of foundation work frequently saves much larger expenditures later. The entire responsibility of determining the method to be used in carrying out a given design is too often left entirely to the contractor, instead of considering the method and final structure desired as parts of the same construction, both of which are essential to a completed whole. Mr. Wentworth.

* Wayne, Pa.

Mr. Wentworth.

The relative amount of yielding in the rip-rap embankment of the large coffer-dam confirms the results of the small-scale tests remarkably well, and the description in the paper gives all the points necessary for use in applying this method to similar work. The difficulties encountered in the construction were surprisingly small, considering the magnitude of the work; and there is evidence of careful preparation in laying out and carrying on the work which is a credit to both the engineers and the contractor.

A coffer-dam of the open type, without cross-bracing, must fulfill two functions: First, it must provide a cut-off wall sufficiently tight to hold back the water and prevent it from opening channels, through or under the dam, which might endanger its stability or cause a seepage flow which could not be cared for by the pumps with reasonable expense. This does not mean that an absolutely tight cut-off is necessary, and, in fact, a water-tight cut-off is seldom obtained. It does mean, however, that large leaks or seepage must be stopped.

The second function which a dam must fulfill is to provide sufficient stability to resist the total pressure of the water. This stability depends on two factors: The total mass, and the internal cohesion of the mass, or its resistance to deformation. In the coffer-dam under discussion, the cut-off consisted of a double row of steel sheet-piles, interlocked and driven to rock, and sealed with a clayey mud. This cut-off was ideal as a stop for percolating water. The rip-rap furnished —in the most compact form of any loose material—the necessary mass, with the maximum stability attainable, short of a coherent structure.

In this case there was a coffer-dam requiring a cut-off wall 68 ft. in height. Engineers must expect to encounter these problems more frequently in the future, and the methods used must meet the conditions. The methods applicable in any given case should be governed by local conditions and the cost of available material. When this work was started, 2 years ago, the cost of steel piling was low, and contractors were seeking places for the cheap disposal of rip-rap from the subway excavations. These two factors undoubtedly had much to do with the type of coffer-dam selected. A third factor which defined the limits of the work was the rock bottom, which made an unyielding support, both for the steel cut-off wall and the rock fill. A variation of any one of these three factors, such as a high cost for the first two or a different character of bottom, might have made some other type advisable. Each of these factors should be given its true weight in considering the use of similar construction in other work.

If the same work were to be done again, with the present high price of steel, and rip-rap being more expensive, some other means might well be considered, such as the use of open concrete caissons sunk by the dredging method and weighted by filling with mud, or

timber caissons similarly sunk and weighted. These two types would represent coherent structures in which the minimum quantity of material is required to resist the pressure, and their use in this or any other case would be entirely a question of relative costs. Mr. Wentworth.

Another dam which should be considered is the ordinary timber sheet-pile and earth-fill type. Such a dam would necessarily have to be placed a greater distance from the bottom of the excavation in order to provide room for the greater mass and flatter slopes required, and also to provide a longer distance for the travel of seepage which might pass a cut-off wall not carried to rock. The latter method would not meet the restricted conditions at the north and south sides of the coffer-dam under discussion, and, in fact, the general method of construction was varied at these points to suit local conditions, large circular caissons being used at the southwest corner, and a single line of sheeting along part of the north side.

These alternates are pointed out in order to call attention to methods which might well be considered in connection with a similar problem under different local conditions.

The paper suggests some thoughts which may not be out of place in this discussion. Harbor development in the United States is now at a point where new methods and types of construction will be required to meet the increased draft of ocean vessels, and to provide for a greater permanence in the design of docks and bulkheads than has prevailed in the past. Timber has been cheap and of excellent quality, and has served its purpose well, for coffer-dam and dock work, for depths of water up to about 30 ft.

Most of the docks in the United States are built of timber, usually untreated. The life of work of this class is short, unless entirely submerged in waters free from marine worms, but it has had the advantage of cheapness and rapid construction, and these have been the deciding factors in a rapidly growing and changing community. In the teredo-infested waters of southern and Pacific ports, creosoted timber and other forms of protection have been a necessity for wooden piles, and, even with such protection, the increased cost and decreased life of dock structures built of this material have made the need of more permanent materials desirable.

For permanent docks in depths of 40 ft. or more, which are now required for the largest ocean vessels, timber is getting beyond its reasonable limit, and the use of concrete or masonry walls or piers, or supporting dock structures by concrete caissons, will probably be the next step forward to meet these new conditions of permanency and depth. These methods are suitable, not only in waters where the teredo is not found, but also for most conditions encountered where a permanent dock is required.

Mr. Wentworth.

Concrete piles have been used in many places infested by the teredo, but they cannot be considered as permanent in waters subjected to ice and frost action, and their permanency in tropical waters is questionable, due to the greater chemical activity of the dissolved alkaline salts in the warm water and the great rapidity with which the steel reinforcement disintegrates when the necessarily thin protecting layer of concrete is broken or punctured.

No engineering work is immutable, and permanency, when applied to harbor work, can only be considered as relative; but it is evident that a new era has been reached in constructing works of this kind, especially in ports like New York. It is a pleasure, therefore, to see the substantial design of the dock walls constructed inside the coffer-dam for the 46th Street Pier, and it is hoped that this work may be a precedent for future construction.

Mr. Wiggin.

THOMAS H. WIGGIN,* M. AM. SOC. C. E.—The author very modestly calls this an unusual coffer-dam. A stronger term might very properly have been used, as will be agreed to, the speaker thinks, by any one who has had experience with such work. The construction itself, after the coffer-dam was completed, and even the methods of building the coffer-dam, were not in themselves so bold, because, though requiring great skill and ingenuity on account of unusual dimensions, they involved methods of pile-driving and excavation which are sufficiently well standardized; but the conception of backing so high and so large a coffer-dam with a mass of rip-rap of comparatively small thickness was certainly one which required boldness, care, study, and sureness, in order to warrant a construction, in that limited space, which did not give opportunity for increasing the strength of the backing, in case it should be found insufficient.

It seems to the speaker that the experiments undertaken to discover the action such a structure might take, deserve more emphasis than the author gives them. They were, apparently, the only quantitative proof of the design. The speaker made a few rough computations, and would be interested if the author would include an outline of such computations as were made by him to justify the proportions of the design. For example, the prism of rock which supported the steel walls on the side next the river, appears to have had a weight of about 200 000 lb. per lin. ft., and the water pressure against it appears to have been approximately 100 000 lb. This would indicate the necessity for a coefficient of friction of about 50% in order that the rock fill should withstand successfully the inward pressure when the coffer-dam was unwatered. The experiments appear to show that the coefficient of friction at the ultimate strength of such a "granular" structure, when subject to sliding pressure, would be considerably greater than 50%, perhaps half again as large.

* New York City.

Mr. Wiggin.

The margin, then, was apparently safe, but not large, and the considerable movement of the structure indicated that just about the right degree of conservatism was used in the proportioning. If the structure had been a little lighter, it might have exhibited signs of distress as the water was lowered inside, and, as there was little more room for additional rip-rap inside, a determination of the next step would have been a serious problem. No such distress occurred, however, and the design was justified, which must have been a source of great gratification to the author and his assistants, and is reason for admiration by the Engineering Profession.

The paper leaves little to be added, though its descriptions are given with a minimum of words. There are a few points, however, which might be explained in more detail. The first is the use of wash-borings and steel-shod piles for the purpose of finding the position of bed-rock. Under many circumstances, neither of those methods would have been reliable. The speaker knows of areas where rock has been located in great detail by wash-borings, but, on excavating, no rock was found; and, similarly, with the driving of rods or piles, it is difficult to know that one does not strike boulders. The speaker presumes that the Dock Department was familiar with the general nature of the shore, and felt confidence in the methods; otherwise, that a few borings of another sort, that is, diamond drill or core borings, would have been made in order to be sure that certain critical points were well established.

It is noted that along the north side of the coffer-dam a single row of steel sheet-piling was used. Of course, two rows give more certainty of cut-off than one, but it would be interesting to have Mr. Staniford explain whether a single row would not have done elsewhere. The principal stability appears to have been derived from the rock fill inside the steel piling, and it would seem to be possible to use a single row and back it up so well with rip-rap as to make the cellular construction unnecessary. The speaker visited the work several times during construction, and noted particularly that the cells of the steel piles along the south line were very much distorted by the pressure. They were no longer cells, for some of them were convexed toward the interior of the coffer-dam on both sides; that is, they had simply been forced over by the action of the outside fill so that they constituted not a row of cells, but two rows of piling with certain cross-diaphragms. Considering the amount at stake, the speaker is not at all surprised that two rows, or even more, should have been thought desirable as a factor of safety; but, after the experience, the point of view as to whether one row would be sufficient for the future would be of interest.

Mr. Wiggin.

It is noted that the steel piling manufacturers have established a strength of about 9 500 lb. per lin. in. for the interlock. This is an important unit to remember in connection with such steel piling, because it has been well established by tests, all giving substantially the same figure, and it will be possible, in many cases, to use such piling locked together as a sort of suspension for supporting pressures.

There are a few places in the paper where the derivation of certain formulas, or, at least, a reference to their derivation, would add to the completeness; for example, on page 505, there is a factor of 57% which presumably comes from the Rankine sine formula for active pressure. The formula on page 508, for coefficient of friction, would be clearer for some explanation.

In one or two places, also, references to slopes are ambiguous, such as a slope of 1.2 to 1, or 3 to 1. The wording leaves doubt as to whether the horizontal is the larger or the lesser figure in the ratio.

In connection with this paper, mention might be made of some steel piling work which the Board of Water Supply did on Staten Island. The Narrows siphon is a 36-in., flexible-jointed, cast-iron pipe, laid across the Narrows, from Brooklyn to Staten Island. On the Staten Island shore, it had to be laid below the level of the bottom of a future dock, and as the material for that dock had not been excavated, it was necessary to carry a trench for a length of about 250 ft. through a firm sandy material varying in depth from about 10 ft. at the toe up to about 50 ft. near the shore, the maximum depth obtaining for some distance. This pipe was laid under water, so that it was not necessary to unwater the excavation, which, therefore, was not a coffer-dam, but merely a sheeted trench with earth and water outside and water alone inside.

The first design considered was the use of interlocking steel piles, laid in straight lines, with wooden rangers and cross-bracing. That, undoubtedly, would have been successful, but it would have required the placing by a diver of a good many pieces of timber for the rangers, posts, and braces.

It was finally proposed to sheet the trench with interlocking steel piles driven in arcs festooned between steel master-piles. The sheet-piles carried the load as a suspension in a horizontal plane between the master-piles, which were held apart by wooden braces. This design differed from that first described in substituting suspension systems for wooden rangers, an arrangement which incidentally eliminated the bearing across grain of braces on rangers, which is a source of weakness.

The sheet-piles were supplied by the U. S. Steel Company, and were 12½-in. and 38 lb. per yd. The so-called master-piles were spaced about 15 ft. apart along the trench; each consisted of a 12-in., 50-lb., I-beam, with a 12½-in. sheet-pile riveted to one flange and a 12-in., 25-lb. channel to the other, to give it additional strength.

Mr. Wiggin.

Many of these sheet- and master-piles were about 65 ft. long. The arcs of festoons were 15 ft. in radius, which, under the assumptions of earth pressure, caused a theoretical tension in these curves of about 3 000 lb. per lin. in., as compared with 9 500 lb., which was the supposed strength of the interlock.

There was difficulty, as was expected, in driving the master-piles exactly plumb and in correct position through the firm material. After they were down, each one was carefully surveyed by locating two points on it at the top as far apart vertically as practicable, and then projecting the line of these points to the position at the bottom. An investigation was then made as to what curvature would really result, making due allowance for the bending of the piles and inaccuracies in the location measurements, and wherever the curvature to be expected was less than about one-half that which was tried for originally, another sheet-pile was put in the arc, thus increasing the middle ordinate of the curve and reducing the stress.

Mr. Thomson.

T. Kennard Thomson,* M. Am. Soc. C. E.—Mr. Staniford deserves the thanks and congratulations of the Engineering Profession for the marvelously successful solution of a most difficult engineering problem. Commissioner Smith also deserves the thanks of every one in New York City and State, as well as of many outside these limits, for his courage, ingenuity, and success in securing for Manhattan the 1 000-ft. docks for which there is such urgent demand.

The author's experiments are very interesting; but, as a rule, it is almost impossible to make such experiments really conclusive, on account of the great difficulty of allowing for all the conditions of Nature. For instance, in a small model, a pile representing a rock fill dumped into place would probably have a considerably smaller percentage of voids than in the actual work, and some of these voids might occur in such a way that the subsequent breaking of a few stones might cause a sudden slide, with injurious results, especially if the stones are coated with slime, which condition, also, could not be obtained in the model. Then, again, the formation of ice might be disastrous, although difficult to make allowance for in the experiment.

A coffer-dam or retaining wall, supporting a fill, built of such proportions as to be absolutely safe under most conditions, will fail completely under a load of semi-liquid material of the same weight and height, due to the fact that it will act as a sort of ram. It is always very difficult to estimate the effect of water on a fill, during or after construction, as the cumulative effect is much greater in some materials than the hydraulic head would indicate.

* New York City.

Mr. Thomson.

In sub-surface work or excavations, it is even much more difficult to estimate the real pressures on a coffer-dam or retaining wall. The materials have been placed, for the most part, by Nature, under great pressures; and are often cemented together by clay or other natural cements, and by roots, stones, etc., so that considerable excavation can often be made without any coffer-dam or bracing. These conditions cannot be reproduced in ordinary experiments, and, in actual work, are often suddenly reversed by the action of springs, rains, etc.

Any experiments, therefore, which are intended to reduce the sections of retaining walls, dams, or coffer-dams below the standards adopted as the result of many experiments, theories, and actual examples, should be checked with the utmost care and foresight.

The author refers to the use of manure for stopping leaks. A very simple expedient, often used successfully in coffer-dam work, where the leaks are small but persistent and troublesome, is to drop an occasional shovelful of ashes or cinders in the water above the leak. This would probably also remedy many leaks in concrete and masonry dams and thus prevent much serious damage from ice forming in the cracks and crevices.

Although too much credit cannot be given to Commissioner Smith and Mr. Staniford for conceiving and constructing this greatly needed improvement, one cannot help comparing it with a surgical operation, for instance, for appendicitis. Both are expensive, and both are absolutely necessary. In this case it was necessary to remove 12 acres of valuable New York City real estate to make place for water.

The number of times such a surgical operation as the removal of the appendix (or city real estate) can occur is limited, of course, as far as the individual man or place is concerned, yet Manhattan needs many more 1 000-ft. docks—in fact, all that can be obtained—but the upper Hudson is not the most accessible place for them.

Six years ago, the speaker was much impressed with the wisdom of the War Department in refusing to allow further encroachment in the Hudson River, in spite of the very urgent need for such docks; he has always felt that there is a correct solution for all problems, if one only has the patience and ingenuity to find it.

In this case, when extension to the west was forbidden, and to the north and east was impossible, the only direction left was the south, it occurred to him that an extension might be made from the Battery to Governor's Island. Such a course, however, would obviously only accentuate a bad condition, for the City is too long and too narrow as it is. Later, the idea occurred to him that the City might be extended from the Battery 4 miles down the bay, with a width of about $\frac{3}{4}$ mile, and that a series of tunnels to Staten Island might

then be constructed. The City Hall would then be in the center of the city, instead of at its southern end. Mr. Thomson.

The next step would be to form a new bay between Staten Island and Sandy Hook, and introduce many other improvements, with which most engineers are familiar. The net result of this plan would be 50 sq. miles of new real estate and 100 miles of additional water-front. Some 6 or 7 sq. miles could be used for a free port between Staten Island and Sandy Hook. This, alone, an eminent banker has declared, would prove to be the salvation of New York.

Hamburg had to evict 16 000 people, as well as place drastic restrictions on many miles of river front; and even Bremen and Copenhagen were much restricted in their efforts to secure enough water and land space for their invaluable free ports. New York, however, can obtain all the land required without interfering with any one's interests.

CHARLES S. BOARDMAN,* M. AM. SOC. C. E.—The speaker wishes to call especial attention to the design of this coffer-dam. Mr. Staniford claims that a precedent for the design is found in the coffer-dams built by the United States Government at Black Rock Harbor, Buffalo, N. Y., and for removing the wreck of the battleship *Maine*, at Havana, Cuba. This is entirely true as to many details, such as the use of long steel sheet-piling, especially in the *Maine* coffer-dam, for the material in which each was constructed was harbor silt below about 35 ft. of salt water. The foundation of the *Maine* coffer-dam was in clay, but the steel sheet-piling for the 46th Street Pier was driven to bed-rock. It is true, also, in comparing the Black Rock coffer-dam, when it is considered that in that structure the steel sheet-piling was driven to bed-rock, although through good sand, clay, and gravel. Both the Black Rock and *Maine* coffer-dams differ in so many other details that neither can be considered a precedent for the design of the coffer-dam for the 46th Street Pier, New York City. Mr. Boardman.

Steel sheet-piling has been used in cellular-form, gravity-type coffer-dams, at Black Rock, Havana, Cape Fear River (Browns Landing), N. C., and Troy, N. Y., but never before as a cellular-type cut-off wall in a gravity-type earth and stone coffer-dam.

The form of the cell or pocket used by Mr. Staniford—with straight partition walls and curved longitudinal walls—has been developed from experience obtained at Black Rock and at Havana. It may be of interest, therefore, to follow this thought more in detail, giving a brief history of these two coffer-dams.

Mr. Staniford states:

"It was expected that the filling within the pockets [at Black Rock and Havana] would make them stable against the external pres-

* Buffalo, N. Y.

Mr. Boardman.

sures, unaided, but in both cases it was found necessary to place an embankment of stone against the pockets, and in the case of Havana, extensive bracing to the *Maine* was also placed, in order to stop the continuous inward movement of the coffer-dam."

This statement is correct, but the stone embankment was placed in each case for an entirely different reason than in the 46th Street Pier, where it forms part of the original design of the coffer-dam. This detail will also be referred to in a brief history of these coffer-dams.

The Black Rock coffer-dam (Fig. 26) was constructed in 1908 and 1909, and consisted of seventy-seven pockets, each 30 ft. square, formed of Lackawanna steel sheet-piling, in lengths ranging from 41 to 54 ft. and driven in straight walls. All the piling was driven to bed-rock, making an average of 33 ft. of penetration through hard materials. The maximum penetration was about 45 ft., at the extreme west wall. Fig. 27 is a view, looking south, showing Pocket No. 40, the northeast corner in the foreground. In the background may be seen the City of Buffalo, the Niagara River, and the International Bridge. This photograph gives a general idea of the location of the ship lock in relation to the river.

The excavation was done by a dipper-dredge having an arm capable of digging to about Elevation — 45 ft., leaving from 3 to 6 ft. of clay and sand over the bed-rock.

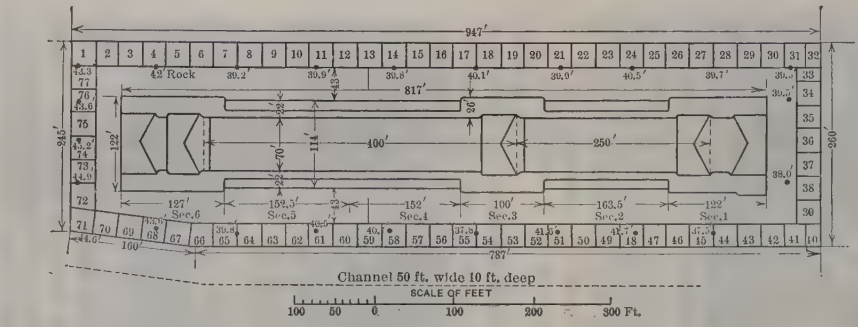
The slopes of the materials forming the embankment by this dredging method were not maintained as prescribed in the Government engineer's plans, and, to re-establish these slopes, rock obtained from other work on the ship canal was placed by dump scows at the toe of the embankment and spread over its top by a dipper-dredge. This rip-rap was placed on the side embankments, but not on the end embankments of the coffer-dam, as will be seen in Fig. 28.

This figure also shows the elimination of the fill at the tops of the pockets against the inside steel walls, which would have formed an active pressure wedge against this wall. Fig. 29 shows the effect produced on the steel sheet-piling wall by the clay filling in the pockets when the embankment is low.

It is readily seen that this piling was thrown into tension at the joints. The maximum curvature of the wall was at a point one-third above the point of penetration, or the top of the embankment, the middle ordinate of these walls at this point being about 3 ft., or about one-tenth of the length of the pocket.

Later, during construction at this end coffer-dam, the toe of the embankment was robbed, causing a slide in the embankment, thus lowering the point of support of the piling. This also created a curved surface of rupture in the fill of the pockets. The steel sheet-piling

Mr. Boardman.



PLAN AND CROSS-SECTION OF BLACK ROCK COFFER-DAM AND LOCK

FIG. 26.

Mr. Boardman.

being deflected further, the result was that the maximum deflection was at a point one-third of the height from the new point of support. The steel channels broke in many of the pockets, the interlocks of the piling in tension resisting this increased loading.

Before this coffer-dam was closed it was seen that the curved longitudinal walls made the better design, and the four pockets, two at each end, left for the passage of the contractor's plant, were constructed in that way.

It was then found that in the fabricated tee-pile the tension strain from these curved walls had a tendency to pull on the heads of the rivets. The greatest movement inward of this coffer-dam at the diaphragm walls, however, was only $\frac{3}{4}$ in., measuring between the tee-piles of opposite walls.

The coffer-dam around the battleship *Maine*, constructed by the United States Government, under supervision of the "Maine Board", composed of Col. W. M. Black (now Brig.-Gen.), Col. Mason M. Patrick, and Maj. H. B. Ferguson, consisted of twenty cylinders, each 50 ft. in diameter, with closing and connecting arcs of ten piles on the outside wall. The cylinders and arcs were constructed of Lackawanna $12\frac{3}{4}$ by $\frac{3}{8}$-in. straight-web sections, forty with 35-ft. lengths and fifty with 25-ft. lengths, spliced to make sheet-piling 75 ft. long.

The original studies for this work show six cylinders, each 50 ft. in diameter on each side of the coffer-dam and five cylinders, 40 ft. in diameter, on the ends, but this was changed later to the plan shown by Fig. 31.

It was planned to fill these cylinders with heavy clay, displacing the 25 ft. of harbor silt which existed on this site below 35 ft. of salt water and over soft clay at Elevation — 60. It was thought that this heavy clay fill, when dropped, would sink to the clay bed, thus raising or displacing the harbor silt and securing cylinders entirely filled with clay.

Time did not permit waiting to deposit the clay by the dipper-dredge, and the fill was made with a hydraulic dredge. This material blanketed the existing harbor silt in the cylinders. When the coffer-dam was ready to be unwatered, it was found impracticable to settle, compact, or solidify this silt, nor could the water in it be withdrawn or pumped out, and this fill, 25 ft. in height, remained in a semi-fluid condition throughout the work.

For this reason the *Maine* coffer-dam was affected by the water pressures after 15 ft. of water was pumped out, and even showed slight movement with every high tide, the result being that the cylinders gradually crept inward at the top, deflecting at Elevation 40 ft. or more below the water level, or where this harbor silt existed as fill in the cylinders. As the cylinders crept inward, the length of the center

FIG. 27.—BLACK ROCK COFFER-DAM, LOOKING SOUTH. POCKET NO. 40, THE NORTHWEST CORNER, IN THE FOREGROUND.

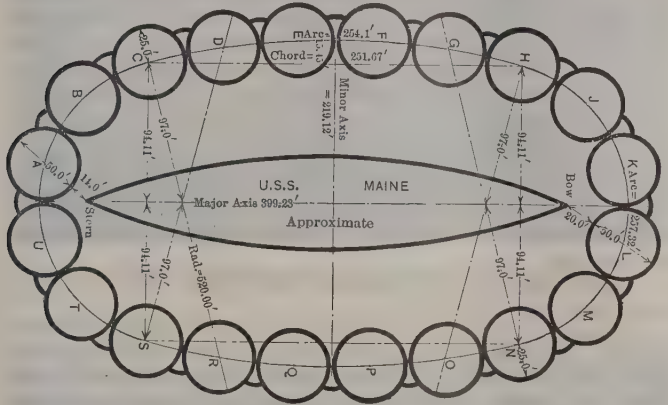
FIG. 28.—BLACK ROCK COFFER-DAM, LOOKING NORTH FROM SOUTHWEST CORNER. WATER 29 FT. BELOW NORMAL LEVEL OF NIAGARA RIVER.

FIG. 29.—BLACK ROCK COFFER-DAM. INSIDE WALL AT SOUTH END. CLAY PUDDLE MOVING PILING TO THE LIMIT OF PLAY IN INTERLOCK.

FIG. 30.—GENERAL VIEW OF BLACK ROCK COFFER-DAM, LOOKING NORTH, AT COMPLETION OF CONCRETE LOCK WALLS.

line of each became shorter, and they took an elliptical form, the area of contact between them increasing. Mr. Boardman.

This was further evidence that, as the diaphragm walls in the Black Rock coffer-dam did not move, this movement of the *Maine* coffer-dam cylinders tended to create diaphragm walls, and led to the conclusion that, when further coffer-dam construction of this type was required, the design could be much improved by adopting the better parts of each, for the form or shape of the cells or pockets. This is clearly exemplified in the plans for the 46th Street Pier coffer-dam designed by Mr. Staniford as steel sheet-pile cellular cut-off walls.



COFFER-DAM AROUND WRECK OF BATTLESHIP MAINE

FIG. 31.

The stone embankment of this coffer-dam was placed during unwatering so as to maintain a balance of pressures and to take the place of the weight of the water and mud removed from the coffer-dam. The base of this stone embankment, being on harbor mud, could not be depended on for a passive pressure in the same manner as the stone embankment for the 46th Street Pier coffer-dam.

The United States Government engineers recognized and used the best features of the Black Rock and *Maine* coffer-dams when designing those for the lock and dams in the Cape Fear River, North Carolina.

Fig. 34 is a plan of the coffer-dam at Browns Landing for Lock No. 2, and shows that three types of wall were used.

Pockets 1, 2, 3, 4, 5, 6, and 7 were of the square type used at Black Rock, except that the 14 by $\frac{3}{8}$-in. section, having a much higher section modulus (7.61), was used with the channel wales and steel rods across the pockets, in order to maintain their form.

Mr. Boardman.

Fig. 35 shows this coffer-dam during construction. The river walls were driven with two floating pile-drivers. A "pilot" pile of pine was driven at the corner of each pocket to support the template used in driving the curved walls.

Pockets 8 to 29, inclusive, but excluding Panel 21, were of a compromise design, these walls being driven on the shore of the river in the original ground. The back wall was driven with the 14-in. arch-web section, and the partition and front walls with the 12¾-in. straight-web section, the steel in the front wall being in tension.

Panels 21 and 30 each consist of a single wall, constructed after the design of bulkhead or dock walls, of steel sheet-piling with steel channel wales and steel tie-rods to anchors. This work was carried out most successfully, the concrete slab being placed under water through a tremie, when the water was near the top of the coffer-dam. The top of the coffer-dam was at Elevation + 32, and the finished lock floor at Elevation + 1.0. The steel sheet-piling was in 49-ft. lengths.

This dam withstood a head of 30 ft. after it was pumped out the first time. Freshets in the river flooded it five times in one season, without any damage whatever. The river was permitted to flow over the top and into the pit without damage. At low stages the leakage was from 200 to 500 gal. per min., thus requiring a small expense for pumping.

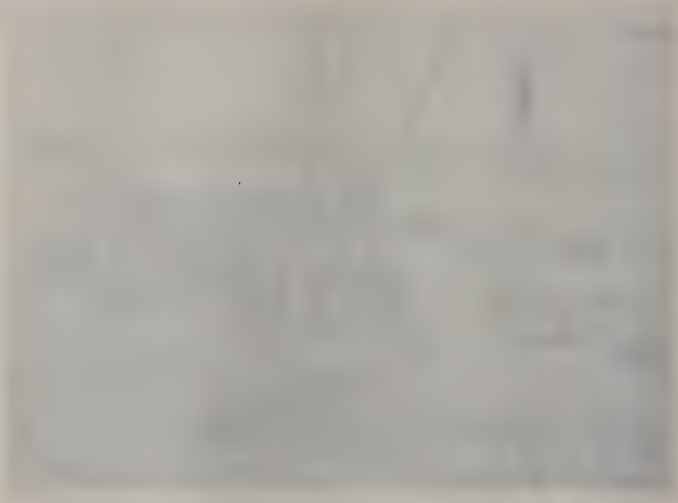
Fig. 36 shows the lock walls under construction, the head of water on the dam then being 26 ft.

In reference to that part of the paper under the heading "Stresses in Steel Sheet-Pile Walls of Pockets", the nature of the material used to fill the cellular-type pocket determines the method of calculating the stresses exerted by this filling on the steel sheet-piling walls. This pressure is transferred to the walls of the steel sheet-piling, and the walls are held intact by the strength of the interlock of this sheet-piling in tension. In 1908—previous to the construction of the Black Rock coffer-dam—the various interlocks of steel sheet-piling were tested physically for this value in tension per linear inch of interlock, and it was established that the Lackawanna steel sheet-piling developed an ultimate strength of 9 700 lb. per lin. in., which was then thought to be sufficiently high to allow such piling to be used for any depth then thought practicable for cellular pockets in a coffer-dam. The maximum stress developed by filling the pockets of the Black Rock coffer-dam, and to be resisted by the interlocks of the steel sheet-piling, was computed for the contractors by J. C. Meem, M. Am. Soc. C. E., and found to be 5 600 lb. per lin. in. of interlock.

Several articles were written by engineers on the stress that would be developed in the interlocks of the piling caused by the filling in

FIG. 32.—CONSTRUCTING THE *Maine* COFFER-DAM, HAVANA, CUBA.

FIG. 33.—THE BOW OF THE *Maine*, WITHIN THE COFFER-DAM.
WATER AT ELEVATION — 10 FEET.



the pockets of the *Maine* coffer-dam, one writer* placing it as high as 7 400 lb. per lin. in. of interlock. Mr. Boardman.

By using the formula adopted to find this tension stress, and making the same assumptions as Mr. Staniford, that the maximum pressure exists at a point 58 ft. below the top of the filling, and also that the weight of the river mud is 80 lb. per cu. ft., with a natural slope of 3½ to 1, the result is 5 480 lb. per lin. in. of interlock. This figure, as will be seen, agrees substantially with Mr. Staniford's 5 200 lb. per lin. in.

In determining the formula to use in this calculation, the horizontal thrust from the material within the pocket was assumed to act perpendicular to the longitudinal piling walls; also, the corresponding unit pressure at any given depth was assumed to be distributed uniformly along the longitudinal walls between the transverse walls of pockets. Therefore, under these conditions, the longitudinal piling wall approaches the curve of a parabola with the maximum tension at the point of support, or at the Y-pile, and its value is

$$t = \frac{\frac{p}{8\,h}\sqrt{l^4 - 16\,h^2\,l^2}}{12}$$

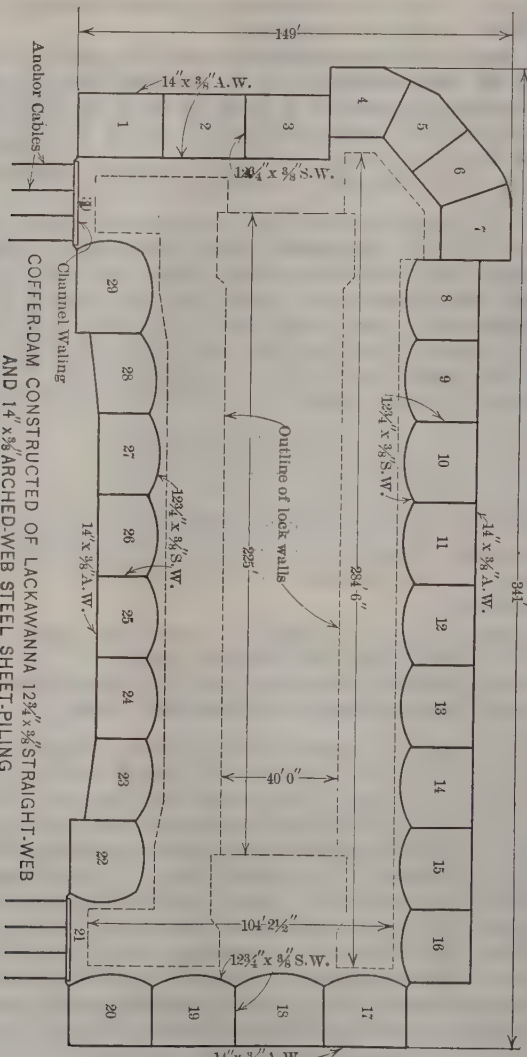
when t = the tension, in pounds per linear inch, in the longitudinal and transverse walls, for the reason that the Y-pile is fabricated with angles of 120°, the components are equal, therefore the result is equal to one of them.

p = the pressure, in pounds per square foot, against the longitudinal walls at the depth considered;
h = the ordinate at the center line of the pocket, in feet;
l = the distance between transverse walls, in feet.

The suggestion has been made that possibly a single wall of steel sheet-piling would have answered for the cut-off wall in a coffer-dam of this type. A thoughtful analysis of this idea will lead to its abandonment almost immediately; first, for the reason that this type of construction, as Mr. Staniford's design shows, requires a rock fill on the inside for stability. Therefore, though it is not impracticable, it is not good engineering to attempt to drive a single wall of steel sheet-piling through this rock fill for such a depth. Again, assuming this cut-off wall to have been constructed before any fill had been placed: this would mean that an assembled single wall, from 60 to 70 ft. in height, would have to stand it vertically, and be secured only by resting on bed-rock and being supported for about one-third of

* *Engineering News*, December 29th, 1910.

Mr. Boardman.



COFFER-DAM CONSTRUCTED OF LACKAWANNA 12¾″ x ⅜″ STRAIGHT-WEB AND 14″ x ⅜″ ARCHED-WEB STEEL SHEET-PILING

Fig. 34.

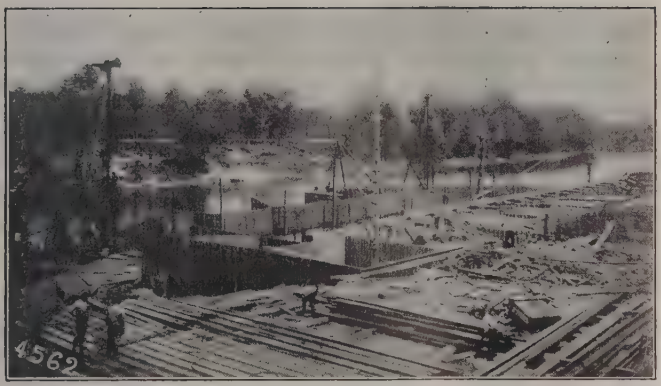


FIG. 35.—COFFER-DAM AT BROWNS LANDING FOR LOCK NO. 2, CAPE FEAR RIVER.

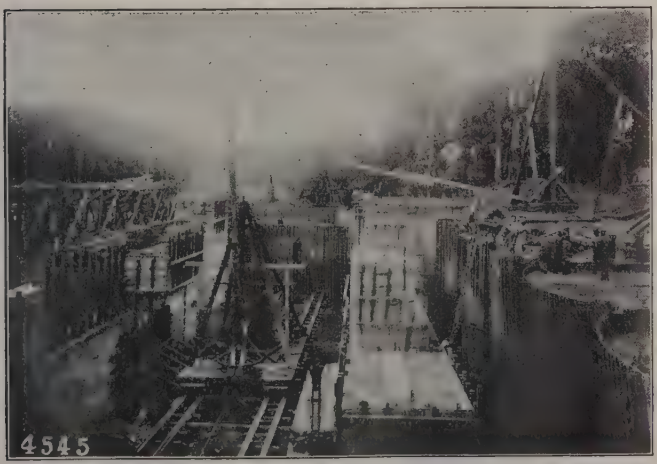
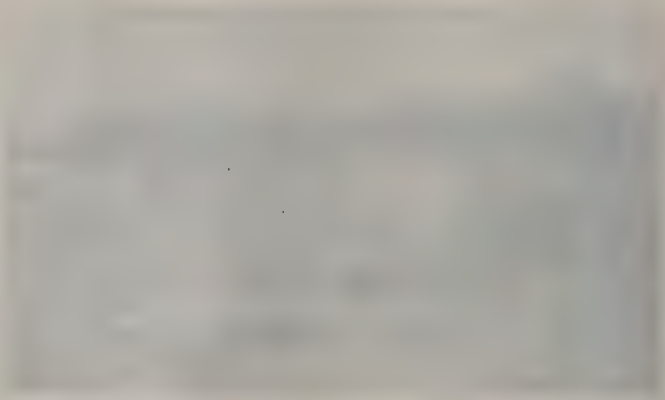
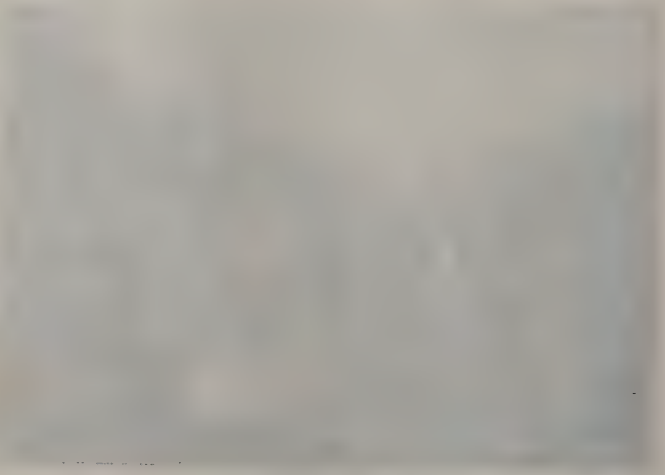


FIG. 36.—COFFER-DAM AT BROWNS LANDING. LOCK WALLS UNDER CONSTRUCTION.





its height in harbor silt. Any one who has assembled a wall of this kind, without first establishing secure supports for it, knows the difficulty in this kind of construction. To support such a single wall of piling until the rock fill could be made on the inside of the coffer-dam and the clay or rock fill on the river side, would require a temporary construction of timber, which would make the cost of establishing this timber support exceed the cost of building the cellular type of cut-off wall as adopted by Mr. Staniford, which, as has been proved, had sufficient base so that it was stable within itself during the filling of the pockets and during construction. It is readily seen that a cut-off wall of cellular type filled with harbor silt is absolutely water-tight, and must have an advantage in this respect over a single wall of steel sheet-piling. Mr. Boardman.

The conditions at the 46th Street Pier, when thoroughly analyzed, show that the bed-rock sloped toward the river and had a very uneven surface, with little clay overlying it, and with harbor silt above it; and that this silt was the only available fill for the pockets. These conditions, and also the extreme depth of the coffer-dam, lead to the belief that Mr. Staniford adopted the best plan for this construction. The remarkable rapidity of its construction and the phenomenal success of the entire coffer-dam are proof positive that this is true.

William M. Black,* M. Am. Soc. C. E. (by letter).—The writer has taken great interest in this coffer-dam, as it was on his recommendation that the rock excavation in the dry was finally resolved upon, and also because there are one or two points made in Mr. Staniford's paper regarding the work of raising the *Maine,* which should be corrected. When the piling of the cylinders of the *Maine* coffer-dam was drawn, it was found, much to the surprise of those in charge, that, as well as could be judged, there had been, in general, no movement of the lower ends of the piles forming the cylinder elements. In general, no movement of the lower end of the piling could be observed at all, but, in one or two of the cylinders, it is believed that some of the piles forming the inside elements of some of the cylinders might have been driven farther down into the clay by the cylinder movements. In general, the movement was a bending of the cylinders without affecting the level of the base. Mr. Black.

F. E. Cudworth,† Esq.—The author and others have discussed the larger aspects of this improvement; the speaker will mention some of the smaller things connected with the work, and will take up a few of the problems which arose in carrying out the practical details of Mr. Cudworth.

* Washington, D. C.
† New York City.

Mr. Cudworth.

the ordering, transporting, handling, setting, and driving, and finally pulling the steel sheet-piling.

Fortunately for the ordering, the Department of Docks and Ferries had complete data, as a wooden pile with a steel shoe had been driven at the corner of each cell or pocket, down to rock. From the rock elevation thus obtained, and assuming a plane surface between the corners, a schedule of piles was made up in lengths varying by 3 in. There were finally more than 3 500 piles in the order. If any one has a similar order to make up, it would be better to use 1 ft. as the variation in length, so as to have to deal with a less number of different lengths.

After giving the matter some thought, it was decided to record the pile data on loose leaves in book form, of a size that could be readily carried in the pocket. The pages contained the necessary information for the rolling mill, the contractor, and the Dock Department, and copies of the books were sent to all parties concerned. As each then had the same information, correspondence and conversation could be readily carried on.

The Lackawanna pile, in cross-section, is symmetrical as to the ends, but not the sides, therefore, the names of "right hand" and "left hand" were adopted, on account of the resemblance of the cross-section to the hand, when looked at from the thumb side. All specials were made by splitting the piles and using the halves, so that the end sections must present a certain outline to suit the closing of a pocket with a fixed number of piles—if an odd number, one outline, if an even number, a different outline—so this end section had to be shown also. The number of piles in the side of a pocket was finally decided on as an even number. This made the three wings of a Y-pile show right-handed in plan. The book solved this problem, as it gave the same information to all persons connected with the work, from the mill to the foreman who drove the piles, and the official who inspected them.

The next problem came in the handling. On the first few cars the piles were in no particular order, different cars even became mixed on the railroad lighter and in handling them from the railroad lighter to the contractor's lighter, or to the scows on the work, so that finding a pile of the required length was quite a task, and the loss of time increased the cost. The manner of handling the piles was discussed by the contractor's force and the railroad company, and the speaker went to Buffalo and arranged to have the piles loaded in the reverse order to that in which they were to be driven. The piles were stacked in groups of about eleven pieces each, with a maximum weight of about 11 tons per group. These groups were put on two cars, three

Fig. 37.—Guide for Entering Pile.

Fig. 38.—Spring Clamp for Attaching Tag Line to Pile.

Mr. Cudworth.

groups sideways and two groups high, separated both horizontally and vertically by blocking, so that the groups were unloaded as units in Jersey City. The groups were then transferred in wire rope slings from the railroad lighter to the contractor's lighter or to a scow, from which they could be lifted one at a time by the contractor's lighter. As each single pile was placed, the next one wanted was found on the top of the group, and from one group they went to the next, and so on. This brought order and economy, whereas, at one time it was thought that a large piece of ground would be required for sorting, with a crane or a lighter to do the work.

The next operation was setting the piling. A lighter was brought from Boston for this purpose, but, in sketching it in on the plan at the time the piling was ordered, it was found that the round corner of the coffer would not admit of placing the lighter so that it could do the work. To accomplish this, and also drive the clusters of land piles for tying back the cut-off wall, two booms were put on a floating pile-driver; this was done in order to avoid bringing in a land driver for a few clusters; one boom held the leads and hammer and the other handled the pile. In this way the land piles could then be driven and the lighter could be taken into the round corner of the dam "head on," in order to set the steel piles. The driver had little freeboard, but this was remedied by closing the forward hatch to keep the water from flooding the hold. This is the only case known to the speaker where two booms were used on a pile-driver. It was also intended to use these booms later for handling two driving hammers at the same time.

It might be thought that the setting of the piles could be done easily, but, with the end of the previously driven pile 20 ft. above the water, and with a pile from 50 to 70 ft. long swinging in the air from a boom, and the pile-driver or lighter moving with the wind and boat swells, it will be realized that it was quite difficult. It is necessary to enter the Lackawanna pile from the end, and as the clearance is only $\frac{1}{8}$ or $\frac{1}{4}$ in., this is not easily done, and there is a chance that the man guiding the pile may lose a finger or two. Light ladders, with a strong hook in the side, so that they would hang to the top of the pile, were provided, and were better than a boatswain's chair, as the man was on something more nearly stationary. The next thing was to steer the pile into place. Various schemes were suggested, and finally a guide (Fig. 37) was made from a piece of piling about 5 in. long, with a swivel-eye for a safety line, so that it would not be lost if it fell overboard. This guide was held in place on top of the pile with light flat-jaw pieces on each side, and had the "thumb," as that part of the lock on the pilings was called, cut out. The weight of the guide was reduced to 11 lb. by cutting. An operator, skilled in

Mr. Cudworth.

handling a lighter, could lower the pile within the distance of the height of this guide; it could then be pulled into place, and would be held by the "finger" of the guide until lowered. This was a time saver and good accident insurance.

The tag line at first was tied to the lower end of the pile, and two men on two lines held the swing of the pile, and helped the guide man. It took time to put on this rope, and more time when the pile was up in the air, to take it off. A little spring clamp (Fig. 38) was finally made, with a swivel-eye for attaching the tag-lines. The clamp was fastened to the pile with a screw bolt having a lever handle, and this could be put on and taken off in a few seconds. The weight of the clamp was 14 lb.

The best day's work in setting, with one lighter, after the crew had been well broken in, was in one case 98 pieces 70 ft. long, and in another case 129 pieces, from 20 to 25 ft. long, in 8 hours. In the latter instance the average was 16 piles per hour, or a pile in a little less than 4 min. Thus, seconds cut off in different ways were important.

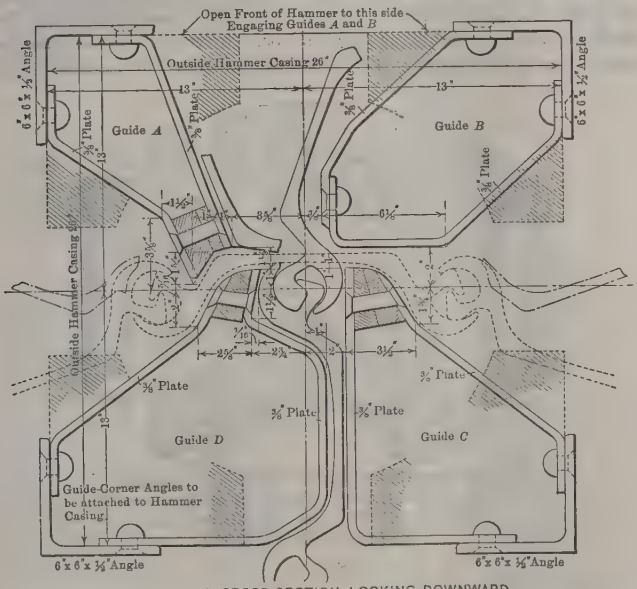
After the setting came the driving. The idea was to use the pile-driver boom and the lighter boom, and do away with the clumsy hanging leads. This is difficult, even when the suspension is from a land machine, but, with a floating machine, moving around and bobbing up and down, it is still more difficult. The first trouble was to get the hammer on the pile when up in the air, the next to keep it there, and lastly (but still important), to keep it correctly lined up with the pile, so that the blows would be in the right direction to do the most good.

Finally, an attachment was designed to be placed on the corners of the hammer. This projected like a double clothes-pin. The hammer was the body of the clothes-pin and the attachments were the prongs. This was built in iron and later in wood. Fig. 39 shows the design for the arch-web style of pile; the arrangement for the straight-web pile section was similar. A double set of prongs—making two slots—was used, so that, by placing the piles in one slot, either three or one could be driven, and by using the other slot, two piles could be covered. The idea was to drive three piles at a time when they drove easily, then two piles, when the driving was harder, and finish by driving one pile at a time, with a follower. This arrangement worked well, and allowed the lifting tackle to be even entirely slacked off, if desirable. In fact, the hammer would work on the pile without being otherwise supported. As shown by Fig. 42, this arrangement was also convenient for taking care of the hammer when not in use, as it saved laying it down on the deck of the lighter.

The problem of pulling the piles was a subject of much discussion, both in the field and in the office. On most of the jobs where piles have been pulled heretofore, with the exception of the coffer-dam

around the *Maine*, and part of the Black Dam, they had been driven in straight lines, or, if curved, the external was greater than the internal pressure, and they were not necessarily in tension in the locks. Owing to the manner in which the cells at 46th Street were filled, the internal was greater than the external pressure, so that the cells probably took a shape somewhat similar to a barrel, that is, they had swelled out in the middle from the pressure of the materials con-

Mr. Cudworth.



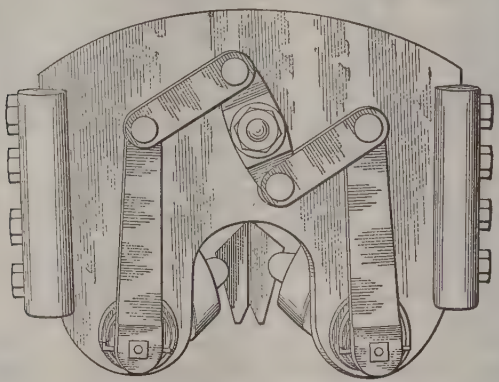
ATTACHMENT TO HOLD HAMMER ON ONE, TWO, OR THREE PILES. (LACKAWANNA ARCH WEB SHAPE.)

FIG. 39.

tained in them, the bottom being held in place by the penetration. The piles to be pulled had been in place for about 2 years, and the earth friction was great. To this was added also the friction due to the tension locks and any additional friction resulting from the effect of the movement of the dam; thus it was difficult to determine the pull necessary to take them out. The locks, however, had been greased when driven.

Mr. Cudworth.

The problem was how to grip the pile and also be able to let go of it when desired, and how to arrange for the pulling. For taking hold of the pile, the Lackawanna Steel Company was kind enough to lend the contractors a 50-ton and a 150-ton grip, which would engage the web of the pile. (Fig. 40.) These grips were used in pulling the piles in the coffer-dam around the *Maine,* and were of the box form, which allowed the pile, as it was pulled, to come up through the inside of the grip. Where these grips had been used previously, the tops of the piles were at about the same elevation; but in the 46th Street coffer-dam they stood at varying heights, as they had been driven to the rock surface, which was quite uneven. The Lackawanna grip could engage a pile



GRIP USED FOR PULLING THE 75-FOOT SPLICED LENGTHS OF LACKAWANNA STEEL SHEET-PILING FROM THE COFFER-DAM AROUND THE U.S.S. *MAINE*, HAVANA HARBOR.

FIG. 40.

about 8 in. below the piles of either side, but when there was more than that variation it was impossible to apply the grip. To overcome this difficulty the speaker designed a grip of such width that it could go in the 8-in. space between two alternate piles and grab one which was lower. This grip (Fig. 41) had 4 by 16-in. side pieces, closed by a toggle and made to open by a single line operated by the engine runner. This was designed for a 60-ton pull. It was to be used in connection with the No. 1 National Pulling Hammer, as shown in Fig. 43.

This apparatus seemed to be strong enough to do the work. The first piles were pulled in the cut-off with a derrick of about 12 tons capacity. This combination pulled the piles which were shorter than about 25 ft., and then there was difficulty. In this case, due to the

Mr Cudworth.

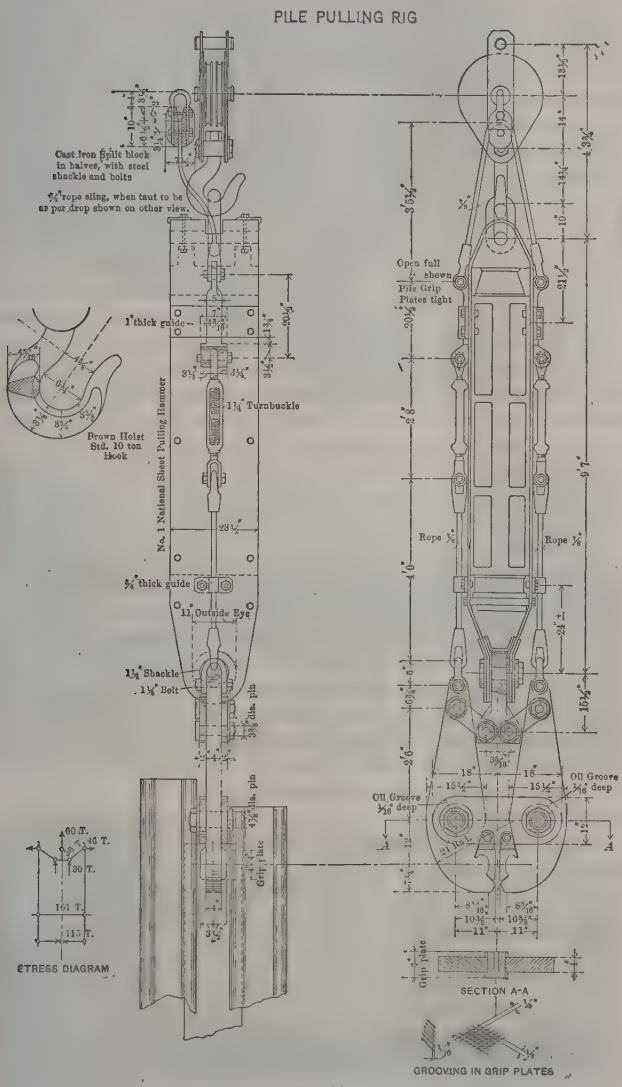


FIG. 41.

Mr. Cudworth.

small capacity of the derrick, the dead weight of the hammer and grip were too great, in proportion to the striking energy of the hammer, and it was found necessary to do away with the grip, drill holes in the sheeting, and attach the hammer directly thereto. At about this time a new grip (Fig. 45), designed by C. S. Boardman, M. Am. Soc. C. E., of the Lackawanna Steel Sheet-Piling Department arrived. This was of smaller capacity, of very much better material, and of lighter weight. It was used afterward on the smaller of the two lighters, having a capacity of about 20 tons, and, after some changes, proved very satisfactory. The large grip (mentioned previously) built by the contractor, was used on the 40-ton lighter, and worked well. The most economical and most rapid method of pulling was when the resistance of the pile was within the capacity of the lighter without the hammer.

Mr. Staniford has described some of the ways in which the piles were pulled; the speaker will mention some of the ways in which they were not pulled: In starting the pulling, in the cells where the piles were long, the hardest work, of course, was in breaking the pockets, that is, in pulling the first piece of piling which had tension in the locks on both sides of the pile and was held by the friction of the soil, stone, and other materials. The first combination tried was a lever and two small hammers which were on the job, using as a lever a 24-in., 100-lb. I-beam, 30 ft. long, putting the 35-ton lighter over the grip and the 15-ton crane on the lever. The beam was bent, and failed to start the pile. A heavier lever was made up, using three 24-in., 1 000-lb. I-beams. The crane at the end of the 30-ft. lever lifted about 8 tons, the other end of the lever rested on the top of a Y-pile, 6 ft. from the pile being gripped. This multiplied the lift on the grip to about 30 tons. The lighter pulled about 40 tons directly on the grip, and, in the direct line of the light pull, there were two No. 3 National hammers inverted for pulling. This was estimated at an equivalent of 15 tons pull for each, and made the total lift on the pile about 100 tons. This rig was used because all the materials for it were on the ground; it failed to start the pile.

The two hammers did not work well because there was no way to synchronize the strokes, or blows.

At this time a No. 1 National pulling hammer arrived and was inserted in the line of pull between the lighter and the grip, in place of the two smaller hammers. This arrangement finally started the first pile in the outside row. After this pile had been drawn a good many of the others could be pulled by the lighter with the grip, without the use of the hammer. An air jet (Fig. 44), with a nozzle similar to that used for a hydraulic jet, was put in operation and served to break up the skin friction with the soil. Then hammer trouble started.

FIG. 42.—HAMMER IN OPERATION.

FIG. 43.—PULLING HAMMER AND CONTRACTORS' GRIP.

FIG. 44.—HAMMER INVERTED FOR PULLING.

Mr. Cudworth.

The band plate, which was of structural material—the best that could be obtained under the market conditions prevailing at that time—did not last very long, and a No. 6 hammer, brought to the job by the McKiernan-Terry Drill Company (Fig. 44), with a band of a different shape was used, as the other bands were expensive and the proper grade of material could not be obtained. The McKiernan-Terry hammer did excellent work.

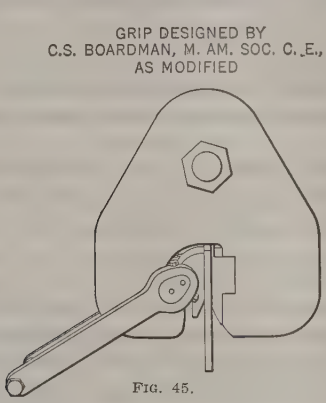


FIG. 45.

The pulling was a fight with the piles from start to finish, there being very few days when a good run could be made. The number pulled by one gang ranged from 0 to 40, a fair day's run was 20. The record was made when 66 piles, not exceeding 25 ft. in length, were pulled with a locomotive crane in about 6 hours. This covers the breaking of the outside row.

The breaking of the inside row with an **A**-frame has been described by Mr. Staniford. An **A**-frame was used because, in dredging out the pockets to a depth of about 10 ft., there was not enough of the dam remaining to carry the weight of the cranes.

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Paper No. 1392

THE VALUATION OF LAND*

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With Discussion by Messrs. W. I. King, Hugh A. Kelly, Edward S. Rankin, T. Kennard Thomson, J. S. Walker, William J. Boucher, Franklin F. Mayo, and L. P. Jerrard.

Synopsis.

In this paper attention is called to the fact that, in the extensive valuation work which has been carried on by engineers in late years, the item of land, in an inventory, has been turned over to real estate dealers. The contention is then made that, even where the services of reliable and well-informed dealers are secured, their appraisals are not entirely satisfactory, and that it is well within the province of the engineer to check the values submitted to him, or even to make the determination of land values himself.

After some discussion of this point, there follows an exposition of the methods and procedure to be used in valuing land, a brief outline of which is as follows:

The principles of land values;
Sources of data regarding land values;
Method of utilizing the data;
Rules for long and short lots;
Corner lot rules;
Plottage;
The cost of acquiring land;
Values based on special utilization;
The valuation of railroad land;
The cost of land appraisals.

* Presented at the meeting of December 20th, 1916.

In recent years there has been a great increase in the demand for valuation of physical property of every description. One great stimulus has perhaps been the organization of public bodies for the purpose of investigating and regulating semi-public corporations, but modern methods of accounting and keeping records are also an important factor. The work of making these valuations has naturally fallen into the hands of engineers, owing to the fact that they, more than any one else, have had to do with the design, construction, and in many cases the selling, of the structures and equipment for which an appraisal is demanded. The various items of inventory—of which, with some exceptions, the most common is land—fall within the knowledge of an electrical, mechanical, civil, or gas engineer.

The item of land appears in almost every inventory, and its importance in appraisal work may be seen from the fact that its value ordinarily constitutes from 5 to 50% of the total value of a utility. It is probably customary for an engineer, in handling a valuation proposition, to feel that the land is somewhat out of his field, and to turn this part of the work over to one or more real estate dealers. In this paper the contention will be made that, even where the services of reliable and well-informed dealers are secured, their appraisals are not entirely satisfactory, and that it is well within the province of an engineer to check the values submitted to him, or even to make the determination of value himself. In some of the greater cities large real estate offices maintain highly organized appraisal departments and, unquestionably, no one is better qualified to make land valuations in certain territory than they are, but it is held that in the country at large the average real estate dealers' appraisals are likely to be in error, due to carelessness, prejudice, or incompetence. It is proposed to show that land values are fixed by more or less definite forces, and to indicate how, by the systematic collection and application of data, a man unfamiliar with land values, as a dealer or even as a near-by resident, may obtain reliable appraisals.

The development of the methods suggested herein should be useful, not only in valuation work, but also in acquiring land for utilities, and especially in cases where controversies arise. In a contest, a real estate broker testifies that he has been a land dealer for a certain number of years, that he has platted and sold land in the vicinity of the tract in question, and that in his opinion the tract is worth a certain sum. In

some cases the very sales he himself has made would, if analyzed, disprove his contention. In what a multitude of cases have not city and county authorities, railroad companies, and other corporations been forced to pay exorbitant sums for land on evidence of this character! In no case may an appraiser gather data enough actually to prove the value of a tract of land, and in the end his conclusions represent his opinion only, but he can present logical reasons for his opinion, and can generally submit enough facts to define a value within narrow limits.

In approaching this subject, it should be borne in mind that land has a value as a result of the operation of definite economic, political, or social forces, and that changes of value take place by reason of similar forces. The location of cities does not occur at random, nor does their growth take place by chance; likewise, the development of farming communities is regulated by their natural facilities. It is the consideration, then, of the factors which affect land values to which the writer will proceed.

The true value of land is the ground rent capitalized. The ground rent is the net income which remains after paying the taxes on the land and the taxes, depreciation, interest, and maintenance on all improvements. The market value may be entirely different from the true value, as it is based, not only on the ground rent capitalized, but on the future prospects of the land's utilization. Farm land differs from city land in that the fertility of the soil, in addition to the advantage of location, is a source of ground rent. Farm land values are comparatively simple, however, and will be treated briefly before taking up city land values, as the latter are much more important.

The value of farm lands depends primarily on the fertility of the soil. Other factors are the topography, water supply, climatic conditions, and the development of the community as regards markets, highways, schools, villages, etc. The advantages of location consist in nearness to cities, railway stations, and schools. The value of farm land is comparatively low and the range of values small. The factors just mentioned vary but little over large areas, and consequently the change in values is gradual. The problem of determining farm land values, therefore, is comparatively simple, because any data which may be obtained are generally applicable over a large district.

The Structural Development of Cities.

The problem presented by cities is much more complex. In the first settlement of new territory, land has no value. A settler has his choice of numerous locations, and occupies any tract that suits his fancy. As other settlers appear, however, there comes a time when there is a difference in desirability between two tracts, due to their location. It is at this point that value arises in the land having a superior location, and it becomes a source of ground rent. As the community grows, more distant lands come into use, a larger area of land has a value, and the value of the best location is forced higher. When the proportions of a city are reached, the grades of land are very numerous, and they are occupied by a variety of utilities. City land is useful only as a site for buildings, and advantages of location govern the values entirely. Any location in the city may be sought by any utility, and the land goes to the one which can make it yield the highest ground rent and, therefore, pay the greatest price for it.

City land may be divided roughly into the following classes as regards utilization:

Distribution.—Retail stores, wholesale stores, railroads.
Administration.—Banks, office buildings, public buildings.
Production.—Factories.
Residence.—Dwellings, tenements, apartments, hotels.

There are other utilizations, such as theaters, hospitals, libraries, etc., which do not fall readily under these classes, but they do not form a large percentage of the total number of utilities. For each of the utilities in this classification there is, with respect to other utilities, a portion of a city within which it is almost forced to locate.

In retailing, the seller must draw the buyer into his establishment. He seeks a location where the largest number of prospective customers will pass, and, consequently, retail stores gather at the heart of the city. Wholesale dealers locate where transportation facilities are best, and as close as rentals will warrant to any retailers whose business they may seek to hold or obtain. Railroads procure sites close to the business center in order to secure passenger traffic, and their freight terminals are located in the wholesale and manufacturing districts. Banks and office buildings are found in the business center, but they can scarcely compete with retail stores for the best locations. Manu-

facturing, to some extent, has the same requirements as wholesaling, but, in addition to railroad facilities, their labor conditions must be considered.

The basis of business values is strictly economic, the land going to the highest bidder, but the basis of residence values has a social element. The wealthy class select the best locations, near parks, easy of access, and free from nuisances. Those of lesser means take locations as near them as possible, and the poorest people are obliged to live in the outskirts of the city or in undesirable places near tanneries, railroad yards, gas plants, etc. Apartment houses are distributed about the same as dwellings; and hotels locate near the retail stores, theaters, and passenger terminals.

The conspicuous feature in cities is the tendency of utilities in the same business to cluster together. The influences which favor one are favorable to another, and, moreover, each establishment endeavors to benefit by the business which other concerns draw. A buyer in a city may visit a number of wholesale houses by traversing a small district, and a jobber located in some other part of the city would probably not receive his attention. In every large city, then, there will be found at the center a business section, certain portions of which are occupied by different groups of utilities, such as retail stores, banks and financial institutions, jobbing houses, etc. Surrounding the business center are sections occupied by the several classes of residence structures, by storage warehouses, factories, railroad yards, and many other utilities. The lines of demarcation between these sections are generally not well defined, as the change from one class of utilization to another is for the most part gradual. It is not uncommon, however, to find two distinct classes of utilization separated by a single street; and barriers, such as a river or a railroad line, often constitute sharp division lines.

The increase in population, the changes in style of architecture and types of construction, and the constant depreciation of buildings cause a continuous movement of utilities from one district to another. In a fine residence section the houses eventually become old and out-of-date, and, as a result, more modern dwellings are built in new districts elsewhere in the city. The old section soon begins to lose its reputation as a fashionable district, the wealthy people gradually move out, the vacated dwellings are used for boarding houses, and the section suffers a decline in land values, even as a rise in values is experienced in the

new district. It is possible for real estate men to gain control of a district, and, by platting it in a suitable manner, making street and park improvements and placing building restrictions on all lots sold, to develop a good residence section. Similarly, by filling in low lands or providing railroad or harbor facilities, a manufacturing or shipping district may be fostered. Frequently the movement of groups of utilities, and consequent changes of land value, are regulated by activities along such lines. In many cases the movement of utilities from one district to another is guided more by the natural growth of the city and the development of its transportation systems, and cannot be influenced as in the foregoing instances. Business property is the least susceptible to such influences.

Business remains on level ground and where topography does not interfere, the retail section moves in the direction of the best residence section. The reason for this is that the residents of such a section represent the heaviest buyers, and their custom is sought by all shops, not alone for the value of their patronage, but also by reason of the fact that all classes go to the shops where the wealthy class trades. The advance of a retail section toward a residence section involves the movement of banks and offices into new districts. The buildings vacated by shops, offices, and banks must necessarily succeed to a lower utility, such as wholesale houses and factories, and these are often followed by tenements and vacancies, so that land once the best in the city sometimes drops to a low value. The point of highest value is constantly moving, and, unless the increase in population is sufficient to overcome the tendency, the land behind this movement suffers a decrease in value.

City Land Values.

From the business center, the main traveled streets project axes of higher value into the outlying territory. As a rule, values grade down regularly from the point of highest value at the business center to the outskirts of the city, and also grade down on each side of the main traveled axes radiating from the heart of the city. This general scheme of values is modified by the location of railroads, bridges, parks, and the different classes of utilities, and also by the topography, kind of platting, extent of street improvements, etc. It is not common to find sharp differences in the values of two adjoining properties. Where a district of low utility adjoins one where land values are compara-

tively high, the low-class land near the boundary is of greater value than the remainder of the district, owing to the proximity of higher values, and, correspondingly, the high-class land near the boundary reflects to a certain extent the low values near-by. Marked changes of value take place, not by sudden variation, but by degrees, over a distance of several blocks.

The total land value of a city depends primarily on the population, but is influenced to so great an extent by the city's wealth and other factors that comparisons between cities of the same population, as to either the total land value or the value of highest priced land, offer considerable variation. It is believed that the range of values in Fig. 2 will cover the conditions in all except a very small percentage of cities. The lower values represent more nearly the normal conditions, and the higher values occur in cities where there is a large transient population or where the business district is restricted. All factors which tend toward the expenditure, in a city, of money which is earned elsewhere, raise the value of business land. For instance, the location, in a city of 25 000 people, of a university with several thousand students would materially increase business land values above those of other cities of the same size. In determining the population of a city, there should be considered such suburbs as are close enough to constitute a part of the city proper. The 1910 census shows the population of Boston to be about 670 000, but the addition of Cambridge, Somerville, Winthrop, and other communities which directly adjoin the city and are closer to the business center than some parts of Boston proper, brings this figure up to nearly 1 000 000. Entering Fig. 2 with this latter figure, the value of the best business land is seen to range from $9 000 to $15 000 per front foot and, when the great tributary and transient population of the city is considered, one would expect to find there, as is in fact the case, values corresponding to the higher figure.

It will be seen from the foregoing that an investigation of land values generally involves a study of the forces at work in a city's development, the land requirements and distribution of its utilities, and the strength and direction of the structural movements which are constantly taking place. The extent of such a study would depend, of course, on the land under consideration. In valuing a single lot in a residence section, such preparation would not be called for, but, in the case of high-priced business land, it is necessary to be informed on these points.

FIG. 1.—MICHIGAN AVENUE, CHICAGO, FROM PARK ROW, IN 1864. THE BUSINESS CENTER OF A CITY MOVES TOWARD THE BEST RESIDENCE SECTION.

For the purposes of this paper the "market value" of land will be considered as the ordinary exchange price that one party would pay another for a farm, city lot, or other tract of land, under normal conditions, that is, when the purchaser is not compelled by circumstances to pay an excessive price nor the grantor to sell at a sacrifice. The market value will not include additional amounts which are often paid for plottage, severance charges, or damages; also, the term "front foot" will be made to mean a strip of land having a frontage of 1 ft. on a street and a depth of 100 ft. measured at right angles thereto. The unit used in measuring city land varies in different parts of the country. In some places it is the custom to use the front foot of 100-ft. depth, or some other depth, and in other places the square foot is the unit. Those accustomed to think of land values by the square foot can readily convert the figures in mind to the front foot as used herein by multiplying by 100.

Sources of Data for Determining Land Values.

In making land appraisals, the following classes of information are available:

Inspection and general study.
Sales of real estate.
Tax assessments.
Rentals.
Opinions of land dealers.
Opinions of land owners.
Character of buildings, as an index to land values.
Real estate for sale.

Inspection and General Study.—In connection with any property, there should be obtained at the outset all information with regard to the dimensions of the tract, the accuracy of the survey, the condition of the title, the amount of taxes and special assessments due, and the existence of mortgages or restrictions as to the use of the land. An inspection is generally necessary in order to determine the extent and condition of street improvements, and the nature of the property as regards topography, filling, protection of water frontage, etc. A general study also involves some investigation of the district in which the property is located and of the city's development and structural growth, as heretofore outlined. A knowledge of general conditions enables an

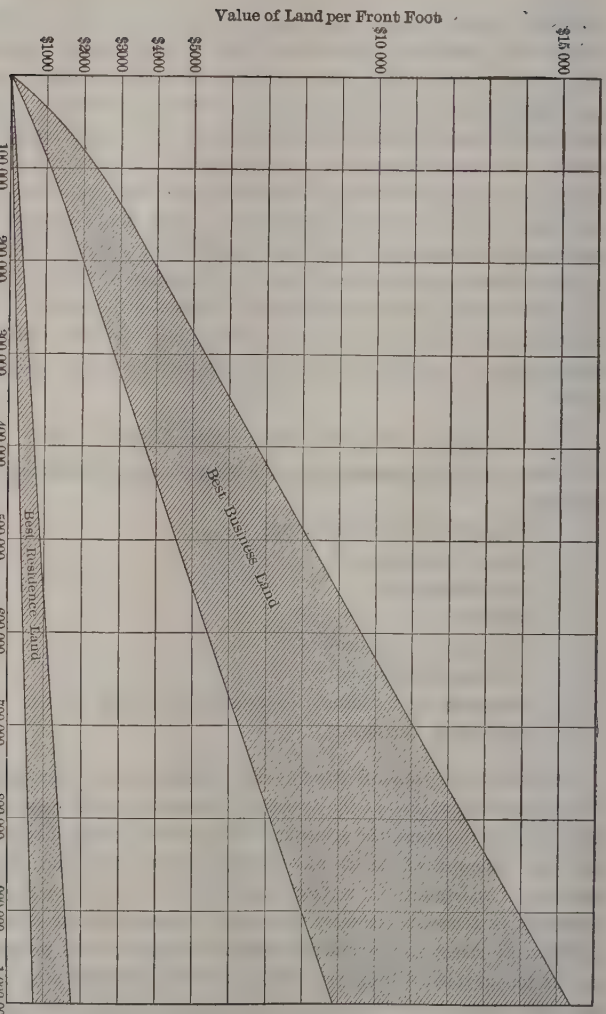

Value of Land per Front Foot
$15 000
$10 000
$5000
$4000
$3000
$2000
$1000
Best Business Land
Best Residence Land
100 000
200 000
300 000
400 000
500 000
600 000
700 000
800 000
900 000
1 000 000
Population

FIG. 2.

appraiser to make use of data which are not directly applicable to the land in question, and often establishes limiting values on a tract of land before a detailed study is undertaken. It is always advisable to be provided with a map showing, not only the parcel of land in question, but the surrounding district for a good many blocks on every side. By recording on this map sales, assessments, rentals, topographic features, etc., their bearing on land values may be most clearly understood.

Sales of Real Estate.—Actual sales of real estate offer one of the best criteria of land value available to an appraiser. Where they can be found in sufficient numbers to form a basis for the judgment, there is no better evidence of market value. In residence districts, where transfers are always taking place, and in the newer portions of a city, where sales are numerous, the work of an appraiser is simplified. After a list of sales has been obtained for land valuation purposes, the appraiser should go to the public record and read the deeds, in order to cull out transfers which do not show true values. Some of the reasons for which sales should be rejected are nominal consideration, material reservations or restrictions, only part interest conveyed, personal property included, conveyance to perfect title, trade, deal between relatives, conveyance to railroad company or municipality, scattered parcels in one conveyance, sheriff's deed, tax deed, etc. These facts cannot always be determined simply by reading the deed, and there are other reasons for rejecting a sale which are almost sure to escape observation by that method. Among such are conveyances by swindlers, fictitious considerations, company and partnership deals, and land contracts. It is advisable, therefore, in important cases, to learn as much as possible regarding the circumstances under which a transfer of real estate is made.

The use of sales is subject to the following disadvantages:

1.—Large numbers of sales are recorded with a nominal or fictitious consideration, and it is often difficult to learn the *bona-fide* sale price. This is more particularly true of high-priced land.

2.—As land reaches high values, the proportional number of sales decreases, so that in certain classes of property such as high-class business land, or dock property where harbor space is limited, the sales data are very meager.

3.—Sales of real estate often include buildings which must be separated in order to determine the sale value of the land alone. In cheap properties many vacant lots are sold, but where land reaches a high value it is not allowed to stand idle, and a sale price is sure to include more or less extensive improvements. In a considerable portion of the high-priced property that comes to sale, the improvements are old, poorly designed, or not suited to the location. These propositions generally involve the wrecking or remodeling of a building, and the apportionment of the sale price between value of improvements and value of land is a doubtful process.

It may be seen, therefore, that sales are more directly useful in appraising cheaper lands, such as ordinary residence sections, where values are low and changes of value gradual, but that their availability decreases in somewhat the same proportion as land increases in value.

Tax Assessments.—Assessments of real estate for taxation purposes are generally considered to be of little or no use as a guide to land values, and it is true that the average city assessment exhibits many irregularities. The assessments are made, however, by men whose particular business it is to study the values in a certain section, and as most of them gain great familiarity with their districts, the figures determined by them are a valuable aid to the judgment. Assessments may be faulty in two particulars: first, in that the different parcels are not assessed on the same basis; and, second, in that the whole district is not assessed at full market value, but at some percentage thereof. The best way to determine whether an assessment is well equalized is to reduce the figures in the section to the same unit, such as dollars per front foot, per acre, or per lot, and plat them on a map. Irregularities show up very clearly on such a graphical representation. As a rule, however, an assessment will be found to be fairly good as regards the comparative value of different parcels, especially where it has been developed during a period of years. Taxpayers are constantly comparing their assessments with those of their neighbors, and, as marked differences meet with complaint until rectified, the tendency is always toward better equalization. It should be noted that not all assessments separate land and buildings, for in many communities the sum of the two appears as one item on the public records. In such cases the assessment

on land alone cannot be obtained, unless, as is often the case, it is obtained from the private records of the assessor.

It is possible, then, where land assessments are available, for an appraiser to make an examination of them and, by the elimination of irregularities and discriminations, determine a fair assessed value for a given piece of land. It only remains, then, to determine what percentage the assessment bears to the true market value. This is done by gathering *bona-fide* sales of property, ascertaining the assessed value of such properties, and computing the ratio of assessed value to market value. For this purpose it is possible to use sales throughout a small area surrounding the tract in question, or a ward, or even an entire city. This method amplifies the use of sales data very materially, for it permits the use of sales which have no direct bearing on the land being appraised, and, moreover, is not affected to such a large extent by the fact that real estate sales involve the value of improvements. Assessments, therefore, when carefully studied and used in connection with land sales, constitute valuable evidence with regard to land values.

Rentals.—If ground rent is determined it is possible to capitalize it and arrive at a value for the land. This process involves gathering considerable data and the use of very good judgment in choosing a rate of capitalization. Ground rent is computed by deducting from the gross rental the taxes on the land, and the taxes, insurance, interest, maintenance, and annual depreciation on the improvements. There are modifications of this procedure by which the gross rental, or the gross rental less certain items, is capitalized, and the rate of capitalization is increased to allow for the items just mentioned which are not deducted. There is a disadvantage in using gross rentals, however, for when capitalized they represent the land and buildings together, and there remains the difficulty of separating the value of land from that of the total improved property. On the other hand, the amount of ground rent as computed may fluctuate greatly, due to the influence of management, and it is necessary to know that a property is well administered, lest the ground rent as determined be too low or even nonexistent. It is also necessary to use current rentals, for rentals agreed upon at some previous time may reflect incorrect values, and this is especially true where land values are changing.

The rate of capitalization follows the same course as the yield on securities, and depends on the safety and permanence of the earning

power. The rate ranges from 4% for the best property in large cities up to 8 and 10% for poor properties in small cities, and even higher for land suitable only for temporary or disreputable utilization. The rate varies in different cities, and, as a rule, is lower in the Eastern States than in the West or South. In a district where land values are increasing, the rate of capitalization is below normal, and where values are decreasing, it runs very high. As previously stated, the use of rentals requires care and good judgment in order to obtain satisfactory results.

Opinions of Land Dealers.—The most common method of ascertaining the value of land is to consult real estate dealers, but, in a large number of cases, their figures are found to be unsatisfactory. The average real estate dealer's appraisal represents an inspection, some thought on such sales and rentals as he is familiar with, and an interview possibly with some other dealer, but no systematic collection and presentation of data. Sometimes great carelessness is displayed in their work. As an instance may be cited the case where several dealers made valuations of a piece of dock property, ranging from $75 to $200 per front foot. Some of them evidently overlooked the fact that the property was not accessible from any street except by passing a long distance over land belonging to another owner, for, in consideration of this fact, the tract was transferred for $66 per front foot. As a rule, the successful real estate dealers' principal assets are nerve and salesmanship. They do not want honestly to know values, but to make sales. They possess an advantage, however, in their knowledge of sale and rental figures, and it is often necessary to gain their co-operation in order to obtain this information. Real estate dealers are generally optimists, and in their opinions often incline to high values, especially in the districts where they are selling lands. Ordinary prejudice, however, does not account for the almost incredible variation which is sometimes found in the opinions of real estate men regarding the same piece of land, which indicates that this class of information may be far from correct, and should be weighed carefully before being accepted as evidence on which to base judgment.

Opinions of Land Owners.—A land owner nearly always has a definite opinion as to the value of his holdings, even though he is not in the market for land, either as a buyer or a seller. This is held to be important information by some authorities on the ground that it is the opinion of land owners which fixes the market value of land. The

probabilities, however, are that the converse is much more often the case, and the opinion of the land holder is fixed by such knowledge as he obtains regarding sales. It is believed that in a great number of cases a land holder is not well enough informed to pass reliable opinions on land values, and that when he is competent it is difficult to obtain from him an honest expression. This class of information can be obtained readily, and should be given some consideration.

Character of Buildings as an Index to Land Values.—The cost of buildings is in a measure an index to the value of the lands which they occupy. In a large proportion of the buildings constructed for business purposes, the cost is about equal to the value of the land. There is, of course, great deviation from this rule, but marked departure is almost always accompanied by financial loss. A cheap building on expensive land is unable to earn an income proportionate to the high land value; and an expensive building on cheap land must necessarily be located outside the district where structures of its class are in demand. On this basis Table 1 has been prepared, showing the value of land occupied by the ordinary types of business buildings, in dollars per front foot, but it should be useful only for very rough estimating. There are numerous instances where there is a great variation in the value of buildings along a street on which lots have approximately the same value, and in such cases the tendency is for the land values to correspond more nearly to those of the highest structures. Exceptions to the general rule are frequent, as, for instance, where cheap structures are erected on good land to serve but a short time, while waiting a better opportunity to make permanent improvements on the property.

TABLE 1.

Size of building.	Construction.	Average value of land per front foot.
2-story	Brick and frame.	$150 to $200
3 "	" "	250 to 350
4 "	" "	400 to 500
5 "	" "	600 to 800
6 "	Slow-burning.	1 000 to 1 500
8 "	"	2 000 to 3 000
10 "	Tending to fire-proof.	3 500 to 5 500
12 "	Fire-proof.	5 500

Real Estate for Sale.—Although there may be no sales of real estate in a district, it is generally possible to find property which is on the market, and obtain an offer of sale. Such an offer throws light on land

values, but should generally be considered as establishing a maximum figure. A party in making an offer usually endeavors to get a high price at first and is obliged to lower his figure before a transaction is finally made. Such offers, consequently, represent values in excess of the market value. They constitute valuable information, however, and have the additional merit of being readily obtainable in the majority of cases.

Determination of Value.

Information bearing on land values may often be found by investigating the expert testimony given in damage cases and other Court actions or hearings, Probate Court records, inheritance tax appraisals, mortgage records, and tax disputes. In valuing land, there may be utilized one or more of the classes of information which have just been discussed. In a district where good sales are plentiful, a brief study of the transfers may be sufficient. As lands reach high values, however, the information bearing on them, not only exists in smaller quantities, but is more difficult to obtain, and often there are few enough data on which to base judgment, even after a careful investigation. The final determination of a market value is ultimately a matter of the judgment of the appraiser, and it is for him to decide in each case what information is required, and just what weight should be given to all the evidence finally obtained.

In summing up it may be said that the important steps in determining the market value of land are as follows:

1.—A thorough investigation of all conditions affecting the land in question, in order that no important considerations may be overlooked, as, for instance, the lack of street connections, in the case already cited.
2.—A systematic and thorough collection of information.
3.—A graphical representation of the data gathered, for by no other means may the relation which values bear to each other be so readily understood.
4.—Determination of value after weighing all evidence.

It should be remembered that the market value of a given piece of land is not a definite sum, and cannot be estimated with absolute accuracy. Several unprejudiced men, all working with the same data, will not obtain similar results. In certain cases it may be possible to

establish a definite market value on cheap vacant lots, but, when property values run into several thousand dollars, reliable estimates will vary 5%, and in complex situations 10%, 25%, or even more.

Long and Short Lots.—Having determined a market value of land, the next problem is the application of that value to lots of irregular size and shape, corner lots, and groups of lots. A market value of dollars per front foot refers to property of average depth, depending on the manner in which the city is platted, and it is frequently necessary to apply this value to a lot either shorter or longer than the average. Several tables have been compiled by different authorities showing the percentage of the total value of lots 100 ft. in depth which is represented by a lot of any other depth, either greater or smaller. A number of well-known rules have been tabulated and are submitted herewith in comparative form. The Hoffman rule appeared prior to 1870, and was based on the assumption that the front half of a 100-ft. lot was worth two-thirds of the total value of the lot. The 4-3-2-1 rule, which is as old, assumed that the front 25 ft. of a 100-ft. lot represented 40% of the total value, the second 25 ft., 30% of the value, the third 25 ft., 20%, and the rear 25 ft., 10 per cent. The Hoffman-Neill rule is a modification of the Hoffman rule, and the Newark rule was developed by the assessing body in the City of Newark. The Somers rule is the work of Mr. Somers, who was long associated with assessment work in New York City. Mr. Pleydell's rule is based on extensive studies in New York City; and the Lindsay-Bernard rule was developed from a mass of data gathered in Baltimore. The Milwaukee rule has been developed during a period of years by Mr. J. H. Leenhouts, connected with the Milwaukee assessing body, adjustments having been made from time to time as additional data warranted. In Milwaukee and Newark separate rules are used for business and residence property. The Davies rule is the work of Mr. William E. Davies, of New York City, and was adopted by him after a study of more than 10 000 sales. Most of the rules are based on a 100-ft. depth of lot, but the Lindsay-Bernard and Pleydell percentages are based on a 150-ft. lot. By a simple computation it is possible to reduce these percentages to the 100-ft. lot basis, as, for example, to find what ratio a 50-ft. lot bears to a 100-ft. lot, according to the Lindsay-Bernard rule, enter their table and divide the percentage for a 50-ft. lot by the percentage for a 100-ft. lot. This operation gives $58 \div 88 = 66$ per cent. This

computation has been made for all values in the rules which are not based on a 100-ft. lot, and recorded in Table 2 for purposes of comparison. Columns (1) to (11) have been platted on Fig. 3. All the rules are not shown here in their complete form, as some of them give a percentage for every foot of variation in depth instead of every 5 ft., and, also, some of them have the percentages worked out to three decimal places instead of two. Percentages for depths of lots other than those shown in the tables may be determined by interpolation, and the application of percentages worked out to one-tenth of 1% is a needless refinement, as market values cannot be determined with any such degree of accuracy.

TABLE 2.—Comparison of Rules for Long and Short Lots.

Depth of lot, in feet.	(1) Hoffman.	(2) 4-3-2-1.	(3) Hoffman-Neill.	(4) Newark, business.	(5) Newark, residence.	(6) Milwaukee, business.	(7) Milwaukee, residence; on 100-ft. basis.	(8) Davies.	(9) Somers.	(10) Lindsay-Bernard; on 100-ft. basis.	(11) Pleydell; on 100-ft. basis.	(12) Lindsay-Bernard.	(13) Pleydell.	(14) Milwaukee, residence.
5	....	....	17	.....	.....	17	7	13	14	10	.....	9	.....	6
10	....	....	26	25	16	28	14	22	25	17	.....	15	.....	12
15	....	....	33	.....	.....	37	19	29	33	24	.....	21	.....	17
20	....	....	39	41	30	44	26	36	41	31	.....	27	.....	23
25	38	40	44	.....	.....	49	32	42	48	37	45	33	38	28
30	....	....	49	54	44	54	37	47	54	43	.....	38	.....	33
35	....	....	54	.....	.....	59	42	52	59	50	.....	44	.....	37
40	....	....	58	64	55	63	48	57	64	56	....	49	.....	42
45	....	....	63	.....	.....	67	53	61	68	61	.....	54	.....	47
50	67	70	67	72	65	70	58	65	72	66	68	58	57	51
55	....	....	71	.....	.....	74	63	69	76	72	.....	63	.....	55
60	....	....	74	80	74	77	67	73	80	76	.....	67	.....	59
65	....	....	78	.....	.....	80	72	77	83	81	.....	71	.....	63
70	....	....	81	86	83	84	76	81	86	84	.....	74	.....	67
75	88	90	84	.....	.....	87	81	84	88	87	87	77	73	71
80	....	....	88	91	91	90	85	88	91	91	.....	80	.....	75
85	....	....	91	.....	.....	93	89	91	93	93	.....	82	.....	78
90	....	....	94	96	96	95	93	94	96	95	.....	84	.....	82
95	....	....	97	.....	.....	98	97	97	98	98	.....	86	.....	85
100	100	100	100	100	100	100	100	100	100	100	100	88	84	88
105	....	....	.....	.....	.....	102	104	103	102	102	.....	90	.....	92
110	....	....	.....	104	104	103	108	106	104	103	.....	91	.....	95
115	....	....	.....	.....	.....	105	111	109	106	105	.....	92	.....	98
120	....	....	.....	108	108	107	114	111	108	107	.....	94	.....	100
125	....	....	109	.....	.....	109	116	114	109	108	112	95	94	102
130	....	....	.....	112	112	110	117	116	110	109	.....	96	.....	103
135	....	....	.....	.....	.....	112	119	119	112	110	.....	97	.....	105
140	....	....	.....	116	116	113	121	121	113	111	.....	98	.....	107
145	....	....	.....	.....	.....	114	123	124	114	113	.....	99	.....	108
150	....	....	117	120	120	115	124	126	115	114	119	100	100	109
155	....	....	.....	.....	.....	116	126	129	116	.....	.....	.....	.....	111
160	....	....	.....	124	124	117	127	131	117	.....	.....	.....	.....	112
165	....	....	.....	.....	.....	118	128	134	118	.....	.....	.....	.....	113
170	....	....	.....	127	127	118	130	136	118	.....	.....	.....	.....	114
175	....	....	124	.....	.....	119	131	139	119	.....	.....	.....	.....	115
180	....	....	.....	129	129	120	132	141	120	.....	.....	.....	.....	116
185	....	....	.....	.....	.....	120	.....	143	120	.....	.....	.....	.....	117
190	....	....	.....	131	131	121	.....	145	121	.....	.....	.....	.....	118
195	....	....	.....	.....	.....	122	.....	147	122	.....	.....	.....	.....	118
200	....	....	130	133	133	122	135	149	122	.....	.....	.....	.....	119

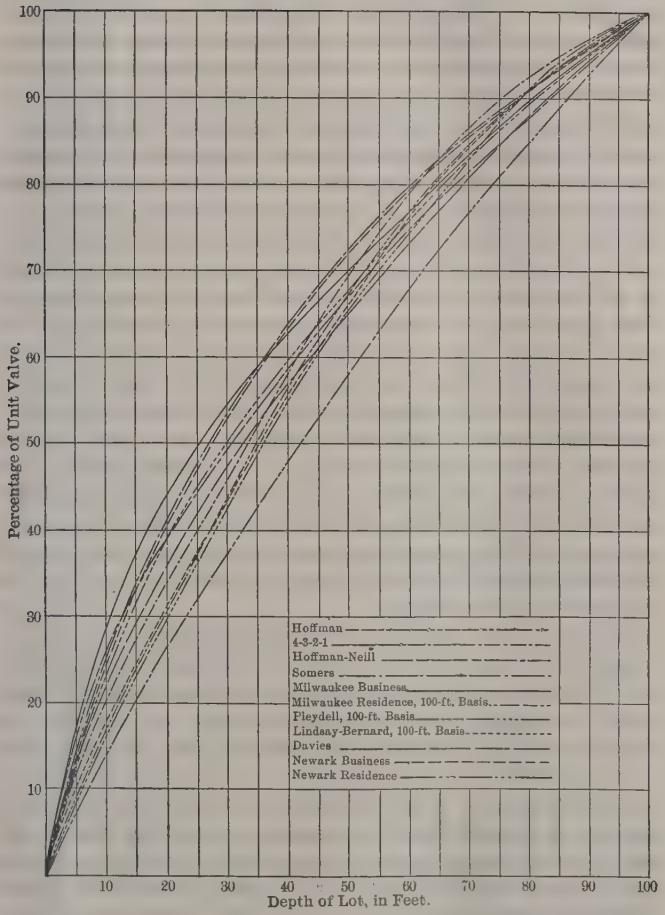

100
90
80
70
60
50
40
30
20
10
Percentage of Unit Valve.
10
20
30
40
50
60
70
80
90
100
Depth of Lot, in Feet.
Hoffman
4-3-2-1
Hoffman-Neill
Somers
Milwaukee Business
Milwaukee Residence, 100-ft. Basis.
Pleydell, 100-ft. Basis
Lindsay-Bernard, 100-ft. Basis
Davies
Newark Business
Newark Residence

FIG. 3.

Of these rules, the Lindsay-Bernard, Milwaukee, Davies, Newark, Pleydell, and Somers are based on actual investigations, and not on assumptions. Of these, the first three appear to represent the most careful study. The Milwaukee authorities have made an excellent point in distinguishing between residence and business property. It is entirely logical that such a difference should exist, and it is easy to understand that in business property a larger proportion of the value would lie in the front portion of the lot, as indicated by the curves. It should be noted that a curve drawn midway between the Milwaukee business and residence curves, and therefore representing average conditions, would coincide more closely with the results of other investigators. The Davies rule is probably more representative of conditions in the large Eastern cities, as it is based largely on data obtained in New York City. It is practically impossible for every appraiser to collect data and formulate his own rule, and it is necessary for him to use the work of others, choosing that which in his judgment appears most reasonable and is supported by the most thorough investigation. It should be noted here that not in every case does a short lot have a greater value per square foot than a lot of standard length. In a tenement district, for instance, a lot too short to be used for tenement purposes would have a lesser value per square foot than a lot of standard length. Such cases are infrequent, however, as such lots are eventually combined with other property, in order that their highest utilization may be attained.

The use of a rule for long and short lots enables one to place a value on a few square feet in any part of a lot, having given the distance to the street. For example, to find the value of a lot 20 ft. deep and 60 ft. from the street, just subtract the value of a 60-ft. lot from that of an 80-ft. lot, as determined from the rule. The application of this procedure to L-shaped lots and lots which have no frontage may be readily seen. Where the sides of a lot are of different lengths they may be averaged, that is, if one side of a lot is 60 ft. long and the other side 80 ft. long, obtain the value of a 70-ft. rectangular lot according to the rule used. Where lots are irregular in shape they may be divided into small squares, and the values of the squares may be determined and summed.

Value of Corner Lots.—A more difficult problem than long and short lots is presented in the determination of corner lot values. Vari-

ous formulas have been devised for computing the value of corner lots, having given the value per front foot of an inside lot on both the main and side streets. It would seem that any formula or rule for determining the corner influence would be inadequate, because the corner lot value is affected by a number of factors, among which are the value of land, the relation of land values on the main street to those on the side street, the width and depth of lots, the height of buildings, the kind of occupancy, and other conditions. In making a city assessment, it is almost necessary to use a rule, on account of the great amount of work to be done; and most of the authorities mentioned have devised a procedure for this purpose. It is felt that none of the rules is adequate, although some of them may be applicable in a majority of cases.

Mr. A. C. Pleydell's rule is to consider the corner as an inside lot on the main street, add 15% and also add 50% of its value as an inside lot on the side street and extended according to the Newark rule. This method is the result of careful study by Mr. Pleydell, but it is understood that he is not prepared to justify it entirely, and offers it more in the nature of a suggestion. A method of determining corner lot values has been worked out by Mr. W. A. Somers, of the Manufacturers Appraisal Company, of Cleveland, the basis of which is not made public. He states that the corner influence in all property generally extends 100 ft. each way from the corner, and he has compiled tables which indicate for any combination of unit values on the main and side streets the value of each 10-ft. square. The values of these squares may be summed to obtain the value of a lot of any shape or size within the 100-ft. corner square. It would seem that any rule applied with such mathematical precision is altogether too inflexible to meet the great variety of conditions which obtain with respect to corner lots.

The City of Milwaukee uses a rule which has been developed by Mr. J. H. Leenhouts, Chief Clerk for 18 years in the city Tax Commissioner's Office, with the co-operation of the Milwaukee Real Estate Board. It is stated as follows: For residence property where all street improvements are made, add 25% of the value of the main street frontage on a width of 30 ft. for corner influence. For business property add the value per front foot (computed as an inside lot and extended according to the Milwaukee rule for business property) of the main and

side streets, divide by two, and take 60% of the result, which determines the value per front foot to be added for corner influence on a width of 40 ft. on the main street. In all cases this method is subject to local conditions and the judgment of the appraiser. As regards business lots, the application of this formula nearly always adds less than 50% to the value of a corner property, as compared with an inside lot, and it is considered that the tendency is to give results which are too low. The Lindsay-Bernard corner lot rule consists in fixing the value of the property as an inside lot on both the main and side streets and adding the two. The sum is taken as the minimum value of the corner, and to it are added all minor factors of value which suggest themselves to an intelligent appraiser. This rule has been tested by its authors in the case of more than 400 sales and rentals, with good results. Although it yields much higher values than the other rules, it is considered to be the most satisfactory of them all.

Many real estate men simply add to a corner lot 50% of the value of an inside lot. Mr. E. H. Gilbert advocates the following procedure for New York City, where nearly all lots have a frontage of 25 ft. To the lot adjoining a corner lot add 10% of the value of an inside lot. To the corner lot add 60% of the value of the lot adjoining the corner. It is probable that a more reliable procedure is represented by a set of rules used by Mr. George C. Olcott, a real estate appraiser of Chicago. In some respects his method is not as definitely outlined as in some of the other rules mentioned, and consequently his system is more flexible.

Mr. Olcott's system of corner valuation is as follows:

"The value per front foot for inside lots of an average depth in any block having been ascertained, the value of the corner lot per front foot may be arrived at with approximate accuracy by the adoption of the following rules:

"On Residence Streets.—To the value per foot on the frontage street, add one-quarter of the value per foot of the side street.

"Thus, if the value of the street on which a lot faces has been placed at $100 per front foot for that block, and the value of the lots on the side street at $80 per front foot, the value of the corner may be safely called $120 per front foot.

"In estimating the value of corner lots on any street, 50 ft. is regarded as coming under the corner influence. If the 50 ft. is divided up into two holdings, the corner value with few exceptions applies only to the immediate corner lot.

"On Residence Streets in Transition to Business Streets.—To the value per foot on the frontage street add one-half of the value per foot of the side street.

"On Business Streets Outside Down-town Center.—To the value per foot on the frontage street add the full value per foot of the side street. The increase, however, should not be less than 25% of the high value unit.

"If a transfer corner, where the intersecting car lines run at right angles for some distance and do not parallel each other partly or wholly, a fair basis to start on is the full value per foot of the values placed on the four converging blocks added together. To this add such a percentage as may be suggested by length of distance traveled and population of tributary territory thereto of said lines. Local conditions, such as license district limits, population immediately around said corner, nearness of factory, church, or school, affect corner values favorably or otherwise.

"Perhaps as accurate results as by any method may be arrived at by estimating the value of all the corners on a given business street, in order to find the relative value of the particular one desired. Select the best corner on the street, then the next, and the next, to the least valuable corner. Get all the data possible, of sales and asking prices. Start from the ascertained facts and increase or decrease proportionately the value of the corners as they have been selected as better or less desirable than the ones from which the data have given a starting point.

"In Factory Districts.—To the value per square foot on the main frontage street, add three-eighths of the value per square foot placed upon the side street. Use square-foot values only in factory districts.

"Down-town Corners.—No rule can be applied successfully to down-town corner values. Each must be determined by itself on a square-foot basis, taking into consideration location, conditions, size, shape, and even occupancy. In large department stores, one square foot is practically as good as another, whether corner or inside, when considered as a whole."

As in the case of rules for long and short lots, every appraiser is not able to gather a great number of data, and must be guided to a certain extent by the work of those who have made extensive investigations. The results of work on corner lot values, however, are somewhat meager and contradictory, and it is advisable to study each individual problem and check, in as many ways as possible, the results obtained by the application of some rule or percentage. A corner lot is more valuable than an inside lot by reason of the increased accessibility, the better possibilities of utilization, and the additional light and air. Increased accessibility and availability include all such

factors as greater facility of entrance to the property, added show-window and advertising space, increased amount of traffic at a corner, and the greater possibilities in designing buildings. If those advantages could be appraised and their value added to that of an inside lot, a check on corner lot values would be obtained.

In the case of business property, where all the factors mentioned must be considered, such a course is scarcely feasible; but it is practicable in simpler cases, such as occur in factory districts, where light and air are the only advantages to be gained by corner locations. The value of the advantage may be measured in such an instance by estimating how much vacant frontage would be required adjoining an inside lot in order to give it as much light and ventilation as the corner lot, and multiplying that frontage by the unit land value. If land has a market value of $400 per front foot and it is estimated that 22 ft. of vacant space would be required to give an inside lot as much light and air as the corner lot, then the sum to be added for this advantage would be 22×400, or $8 800. The estimated additional frontage, of course, should not exceed the width of the side street. This method is modified by a number of factors, such as the height of buildings, the shape and size of the lot under consideration, and the possible design of improvements as regards artificial lighting or the disposition of courts and light-wells.

As a rule, corner influence does not extend across ownership lines along the main street, and property adjoining a corner lot on the main street is of little higher value than any other inside frontage. Only in so far as it is located close enough to the corner to benefit by the increased street traffic there does it have any increased value whatever. Along the side street the corner influence generally extends beyond the corner lot, the value of frontage grading down regularly from the main street corner until the normal market value on the side street is reached. As light cannot be utilized more than 40 ft. from the lot line, the added value due to it should not attach to property more than that distance from the corner. Moreover, if the corner lot is narrow, the light advantage cannot be completely utilized, and is of less value. Corner values in residence property are of much less importance than in business property. In a district where each building is on a fair sized lot with plenty of space around it, the corner has practically no additional value beyond that which is due to the additional street im-

provements; but, in more crowded sections, where there are flats and apartment houses, the corner lot may be worth from 25 to 50% more than inside lots.

Many of the large real estate and appraisal offices have their own rules for handling questions of long and short lots and corner lots, and guard them carefully as office secrets. It is hardly possible that their private studies should yield information varying to any great extent from that which has been discussed herein, except in cases where it applies to peculiar local conditions. A number of the most difficult cases in determining land values are encountered in appraising corner lots. Every corner lot presents a different problem, and, after general rules have been applied, each case should be studied with regard to its peculiar features.

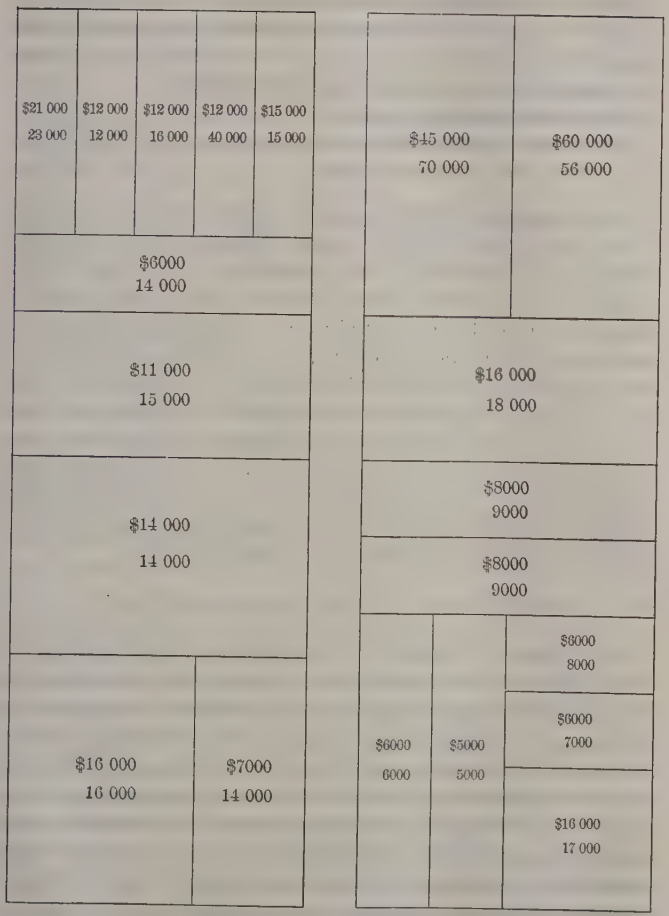
Plottage.—In certain classes of property, where several lots or blocks, or an unusually large frontage, are held under one ownership, a tract of land may have an additional value by reason of its being thus held. Where such conditions prevail, the tract is said to have plottage, and, where the land is more valuable by reason of such conditions, to have a plottage value. Although a piece of land may have an unusually large area, it does not have plottage value unless it can be occupied by some utility which requires an extra large site. If a tract of six or eight adjoining lots is available only for dwellings it will have to be broken up into single lots in order to be utilized, and consequently has no additional value; but, if it were owned by a traction company, and used for a car-barn site in a district where tracts of equal size were not readily available, it would have a plottage value. When a market value for such a tract has been determined, it is customary to add a percentage to cover the plottage value, and 10 and 15% are commonly used. Plottage value exists by reason of the fact that it is necessary to expend amounts in excess of market values in order to gather several adjoining pieces of land under one ownership, and some measure of the amount to be added for this value can be ascertained by estimating what additional expense would be incurred in gaining control of several adjoining properties held under separate ownership. In acquiring land, under such circumstances, it is not always possible to condemn, but it is possible to obtain options and get the proposition lined up so that the owner of one parcel cannot demand a prohibitive sum after money has been invested in several surrounding tracts. Even with careful buying,

however, the grantee is generally obliged to pay an exorbitant price for one or two lots in order to close a large deal. Such a transaction, however, is not made unless a considerable portion of the tract can be obtained at a reasonable market value, and though a double price may be paid for some pieces, the percentage above market value, computed on the entire purchase, is not apt to be excessive.

Fig. 4 is a diagram of a group of lots purchased by a large company for a home office building site, and is a fair example of the manner in which such a property is acquired. Twenty parcels of land were purchased, constituting an entire city block. On each parcel is shown the estimated market value and directly under it the sum paid out for that piece of land. The market value of the entire block was $302 000 and the total cost of acquiring the twenty parcels was $384 000. The excess over market value was $82 000 or 27%, but, nevertheless, the deal represented a very good piece of buying, and the total tract could be said to have a fair plottage value of $82 000. This instance is unusual, a much more frequent case being where three adjoining lots are worth $2 400 each and the plottage value for the three together is estimated at $800. Plottage value may be said to range from 10 to 25% of the market value, the percentage decreasing as the value of property increases, and increasing in proportion to the number of lots or parcels constituting the tract.

Cost of Acquiring Land.

Considered from a different viewpoint, plottage value is simply one of three items which may be called collectively "the cost of acquiring land", as it represents an amount in excess of market value which it is necessary to pay out in order to purchase several adjoining pieces of land. Another item of this group is the cost, in connection with purchasing land, of buying and removing buildings or other improvements which are on the land but are of no value to the purchaser. There are numerous cases in connection with land purchases where large and serviceable structures have been wrecked to make room for a different type of improvement, and, even considering the salvage, the expense involved is often considerable. This cost, of course, is subject to no regular influence, and must be determined under the conditions existing where it is encountered. The third item of the cost of acquiring land is the actual expense in connection with making the purchase, and composes one or more of the following sub-items:



Upper Figures=Market Value
Lower Figures=Purchase Price

FIG. 4.

Expense of searching for and choosing a site;

Salary or commission of men engaged in buying land;

Field expenses of men engaged in buying land (railway, hotel, livery, etc.);

Expense of legal services and examinations of title;

Fees for recording instruments;

Expense of registering title in land courts;

Surveying and setting monuments.

In most cases the purchase is placed in the hands of a real estate broker, and his fee is the only expense. Brokers generally charge as a commission a percentage of the consideration, which varies in different localities, depending often on the rules fixed by real estate exchanges and other brokers' organizations. The cost of abstracts and examinations of title varies according to the condition of the title, and bears no relation to the value of the property. In some cases, where the title is involved, it requires litigation and the expenditure of large sums to clear it. Such cases cannot be estimated, and the expense must be obtained from actual records. Ordinarily, the expenditures mentioned under the third item of the cost of acquiring land vary from 2 to 8% of the market value of the property, the lower percentage being more likely to occur when land reaches high values.

Different Theories Regarding Land Values.

In the valuation of a parcel of land, there may be determined two values: the market value and the cost to acquire the latter, representing the cost of obtaining plottage, the cost of removing improvements not required, and the expense in connection with purchasing. The two values may be determined for two periods, that is, at the date when the land is purchased and at the date when the valuation is made. At the date of purchase both the values represent an estimate of actual expenditures. At the date of valuation the market value may be much higher, due to an increase in values; and the cost to acquire will be a speculative value, also increased over the original cost, as it is to a large extent proportional to the market value.

Assume the history of a property to be as follows: Four lots having a market value of $3 000 each were purchased by a company for the sum of $14 900; buildings of no value to the company were purchased

and removed at a net cost of $4 700; and the expense of purchasing the property was $500. The total expenditure was $20 100, of which $12 000 represents market values and $8 100 the cost to acquire. Assume, further, that 20 years later the lots have a market value of $6 000 each; that a careful estimate of the amount to be added for plottage is $5 000; that the net cost of removing the buildings is placed at $11 000; and the expense of purchasing at $700. What is the value of the tract of land?

Date.	Property.	Market value.	Plottage.	Net cost of removing buildings.	Expense.	Total.
1896	4 lots.....	$12 000	$2 900	$4 700	$500	$20 100
1916	Same property..	24 000	5 000	11 000	700	40 700

The investment cost is $20 100. The capital charge (allowing for the increase in land value) is

$$\$24\,000 + \$2\,900 + \$4\,700 + \$500 = \$32\,100,$$

or, if the increase in plottage value is also included,

$$\$24\,000 + \$5\,000 + \$4\,700 + \$500 = \$34\,200.$$

The sale price to another company would be $24 000 + $5 000, or $29 000, but the replacement value would be $40 700. The foregoing is an extreme case, and is given to show the variation in valuation results, worked out under different theories or for different purposes. In public utility regulation cases, there is a strong tendency, in land valuation at the present time, to use the market value as of date of valuation, and add the cost to acquire as of date of purchase. This is in conformity with a theory which allows a utility to capitalize the increment in land values, but does not consider that it is entitled to an increment, more or less speculative, to expenditures in connection with land purchases.

Values Based on Special Utilization.—In frequent cases a tract of land is held to have an exceptional value because it is particularly adapted to the purposes of the utility by which it is occupied. A steam-power plant may be on the bank of a river where plenty of water is available for heating and condensing purposes, and the conditions may be such that no other site with equal facilities is to be had. The position often taken in such cases is that the location has a peculiar advantage for steam plant purposes, which it would not necessarily have for the purposes of a wholesale house, a railroad yard, or some

other utility; and, as a similar location cannot be obtained, it has a value in excess of the market value of adjoining lands. This additional value may be based on the extra cost of providing a tunnel from some other location to the river, or on the saving effected by the existing water supply over a well supply. A further step in this direction is to give land a value based on its highest utilization, even though such utilization has not been accomplished. A notable instance is the case of certain islands in the Upper Mississippi River which were taken for public use. The jury assessed their value as waste land, and, for any except boom purposes, at $300; but, in view of their adaptability for the formation of a boom across the river, at more than $9 000. The Court found the owner entitled to the larger sum, although the islands were not at the time, in fact, used for boom purposes, but were capable of such use. Instances of this kind are mostly encountered in land damage and land condemnation cases, and are based on the wording of the laws governing such cases, which state in brief that when a man's property is taken from him he shall be recompensed to the extent of its full value for its most valuable use. High values, in such instances as the steam plant location, however, are based on the theory of the cost of replacement of the property. In the majority of these situations, neither was the actual cost of the land excessive, nor could the land be sold to another utility for a high figure; and extra value is claimed simply because, if the power company did not own the site, it would require great expenditure to obtain one which would be as suitable.

The method of handling such cases depends on the theory under which the valuation is undertaken. If the basis of the work is "what would it cost us to replace this land if we did not own it", or "what expenditures would be entailed if we could not obtain it", it is easy to justify extreme values. Consideration should be given, however, to factors which sometimes modify these values. A small rocky island in a river may be said to be very valuable to a water-power company as an anchorage for a dam, but, in valuing the entire property of the company, the fact that a high value has been placed on a bit of waste land increases the theoretical cost of developing the water-power, and, when the value of water-power rights is computed, they will be found to have a lesser value to exactly the extent that the theoretical cost of development has been increased by placing a high value on the

island. Such an island may save many dollars to a railroad company as a footing for a bridge pier, but, if it has been necessary to change the grade or alignment of the road in order to cross the river at that point, the saving may be greatly reduced. Even though the island were directly in the path of the railroad line and could be utilized for a bridge foundation without any extra expenditure to make it available for that purpose, it is questionable whether the land would have any but a nominal value. The island is only a natural feature of the territory crossed by the railroad, and to claim a high value for it is akin to placing a higher value on railroad right of way in a level district than in a hilly one, where land has the same value for farm purposes, but where earthwork and curves make railroad construction and operation more expensive. It is evident that appraising land on the basis of its value for certain uses may be carried to absurdities.

To return to the case of the steam-power plant, previously cited: There are in every large community utilities which require facilities for coal and water supplies, and, if locations with such facilities are limited, the competition is bound to develop a stiff market value for land of that class. On the contrary, in situations where the plant is on land which, if not used for power-plant purposes, would be occupied by a utility which would not take advantage of the special facilities offered, the market value of the land should be based on that class of utilization. Sometimes a utility, in buying a piece of land particularly suited to its purposes, pays a large sum for it when no other utility is competing for the location, simply because the owner recognizes that the purchaser is especially desirous of obtaining it and holds to a figure above the market value. The utility may want the land because it has special facilities for coal and water supplies or because it adjoins land already owned, but in either case it is simply a circumstance that confronts the purchaser, and the market value of the land is not increased thereby. Generally, the sum paid for a piece of land having a special value to the purchaser is not excessive, due to the fact that the seller frequently considers the value from the viewpoint of his own utilization of it, and does not ask himself "what is it worth to the purchaser". A railroad company requiring earth for a large fill may acquire a particularly suitable piece of land at the market value, the seller having in mind only its value as farm land and not considering that the purchaser wants it for filling

material and would have a long haul if the material were obtained elsewhere. Under such conditions, the purpose for which property is wanted is concealed, and, where the identity of the purchaser would suggest it, the transfer is made through a third party.

In concluding this phase of land values it may be said that the cost of replacement as applied to land may lead to extreme results, and furthermore, it is a theory which is considered by many authorities to be unsound for most appraisal purposes. If a market value is to be determined for land which has special advantages for certain utilizations, it should not be abnormally high unless there is competition for the location based on its peculiar facilities.

Value of Railroad Land.—The valuation of railroad lands is a problem which has confronted appraisers in recent years. Such valuations are generally made by employing men familiar with land values, and they gather sales and other data regarding the value of land along the railroad right of way, and use the information collected, supplemented with personal opinions, assistance from real estate dealers, and notes from the company's records, to fix the values. Railroad lands are so extensive, and include so many grades of property, that an appraiser is not warranted in going into details to the extent that is customary in making ordinary land valuations. Not only would it be too laborious a procedure under most circumstances, but there would be little net gain in the accuracy of results. By using a rougher method, any individual tract of railroad land may be given a market value that is in error, but, if all inaccuracies are compensating, the net error throughout a State, or even in one city, is a very small percentage of the total value.

The method of using tax assessments which has been outlined heretofore is particularly applicable to the valuation of railroad lands. Maps are first provided showing the railroad lands and also the adjoining property for several blocks on each side of the right of way and terminal grounds. Owing to the nature of railroad lands, the units, such as lot values and front-foot values, cannot be readily applied, and it is necessary to use the value per square foot. In order to compare and apply the assessments on adjoining lands, they must be reduced to the square-foot unit, if they do not exist in that form in the public records. The assessments are recorded on the map, and the right of way is then inspected in the field in order to determine its relative value

as compared with adjoining lands. In the light of the inspection and the data recorded on the maps, the railroad lands are given an assessed value which is subsequently converted to market value by applying the ratio which has been found to exist in that particular district between market values and assessment values. The market value determined for any given piece of railroad land may be slightly incorrect, but the net result for a city is accurate, as no cumulative errors are introduced. This method may be used in either city or country districts, and is very convenient where good sales are scarce, which is generally the case.

Cost of Acquiring Railroad Land.—In acquiring railroad right of way, many expenditures are incurred in addition to paying the market value of the land purchased. Among the more important may be listed the following:

1.—Wages and commissions of men engaged in purchasing land;
2.—Field expenses, traveling, livery, board, etc., of men engaged in purchasing land;
3.—Printing and advertising, notices, etc.;
4.—Abstracts and examinations of title;
5.—Fees for filing and recording documents, etc.;
6.—Condemnation and legal expenses;
7.—Purchase of buildings, orchards, crops, etc., on land required for railroad purposes;
8.—Damages to adjoining property;
9.—In addition there are the amounts paid out which represent no value received or damage inflicted, but merely the sums which it is possible for land owners to extort from a railroad company under the conditions which prevail in buying railroad lands.

The first six items are small, and range from 4 to 12% of the total amount paid for land. The smaller percentage obtains in the case of a large railroad having a land department and a purchasing organization, and the larger percentages are found in the case of small electric lines acquiring only a few miles of right of way, and using only a temporary organization for the purpose. These items are of minor importance, however, and the cost of acquiring railroad land is determined chiefly by the magnitude of Items 7, 8, and 9. These three items are not

governed by any established laws or ratios, but are controlled to a certain extent by the following factors, and it is impossible to determine the extent of any one influence separately:

1.—Value of Land.—As the value of land increases, the proportional cost to acquire decreases. In buying cheap lands, the railroad companies do not make the effort in purchasing that is made in cities, where the investment is larger, nor are they so likely to resort to condemnation, for it is sometimes cheaper to pay five or ten times the market value of poor lands than to undertake the expense of condemnation.

2.—Ownership.—The class of ownership and the area of land under single ownerships is a factor. In some of the lumber States there are thousands of acres of cut-over timber lands held by single lumber companies, and it is possible to obtain large portions of a right of way at reasonable figures by dealing with only a few such land-holders.

3.—The extent to which a community is built up obviously affects the matter of damages in both city and country lands. In undeveloped territory, it is possible to buy land cheaper by reason of the fact that land-holders are desirous of obtaining railroad facilities. A new line through districts already well supplied with railroads, however, obtains no such favors.

4.—The direction of a railroad line has some effect on the extent of damages. A line running diagonally across the country cuts all farms into triangular pieces, but a right of way which parallels the section lines, preserves the shape of fields and causes less damage.

5.—One of the chief factors in the cost of acquiring railroad lands is the skill of the company's buyers. There are instances where right of way has cost a railroad company 50% more than it should have, owing to poor buying.

There are many other influences which are of a minor or local nature.

The only way to determine the cost to acquire a given strip of right of way is to learn from the company's records what was actually paid out in acquiring it and deduct from that sum the estimated market value of the land. Investigations of this kind show that the cost under

different conditions varies tremendously, ranging all the way from 5 to 600% of the market value. If it is necessary to make an estimate of this cost and no records are available, a study should be made of the results of purchases of railroad lands in near-by territory, under similar conditions to those affecting the right of way under consideration. If such a guide is not to be had, an estimate is largely guesswork. Although the variation is great, the average of the cost to acquire land for railroad purposes in a number of instances has been from 50 to 100% of the market value in cities and from 200 to 300% of the market value in country districts.

Cost of Land Appraisals.

The cost of making appraisals of property generally runs from one to two-tenths of 1%, and, as a rule, the work done on the land should not be more expensive than that on other classes of property in an inventory. Real estate men charge fees for appraising land which range as high as one-quarter of 1% for city property and one-half of 1% for suburban property. The cost of valuing the land of a utility should increase according to the number of scattered parcels to be handled, and should decrease in proportion to the increase in the value of lands to be appraised. It may easily require less effort to appraise a single parcel of land worth several hundred thousand dollars than to place a value on several parcels, the aggregate value of which is not nearly as great.

DISCUSSION

Mr. King. W. I. KING,* ESQ. (by letter).—There is a dangerous and time-honored economic fallacy hidden in the sentence "The true value of land is the ground rent capitalized". After two thousand years of search for the "true value", economists are in agreement that no such thing exists. Furthermore, "ground rent capitalized", unless the phrase is broadly interpreted, gives no kind of value of any significance.

Every prospective buyer or seller of land has in mind a maximum price which he will pay or a minimum price which he will take. These subjective prices are finally based on the beliefs as to the probable future income which may be expected from the land. In other words, each person, using his own rate of interest, discounts these anticipated future rentals and finds their entire present worth. With their subjective prices in mind, bidders enter the market, and, by the well-known principles of supply and demand, fix the market price for the land.

At some periods, nearly every one becomes unduly optimistic as to the future outlook; at other times pessimism reigns. This change in the point of view of the majority causes land values to fluctuate widely on the market. If these values are plotted, it will be noticed, however, that there is a rather definite trend in a certain direction. This trend records values arising from the normal consensus of opinion, and may be said to represent normal land values; but value is always based on anticipation, and, until the veil is rent from the future, no man will ever be able to state correctly that any figure represents the true value of the land.

In a progressive community, land values usually stand well above the capitalized present net rentals, because people do not expect that rentals will remain constant, but rather that they will rise. To correct for this error by varying the rate of interest used in capitalizing the current rent is, at the best, a very crude method of adjustment. The writer has described, in "The Valuation of Urban Realty for Purposes of Taxation",† ways of solving the problem which appear to be more logical.

The writer notes, also, an apparent misconception of the word "economic" as used on page 586. The demand of wealthy persons for pleasant residence sites is certainly as strictly economic as the demand of a railway for a site for a new depot or roundhouse.

Mr. Jerrard's discussion of such topics as the movement of the business section, "plottage", "cost of acquiring land", and "values based on special utilization", is both able and interesting, and appears to be better developed than any writings on the subject which the writer has previously read. The author is to be congratulated on the scientific and scholarly nature of the paper as a whole.

* Asst. Prof. of Political Economy, Univ. of Wisconsin, Madison, Wis.

† *Bulletin No. 689* of the University of Wisconsin. May be obtained from the Librarian of the University.

Mr. Kelly.

HUGH A. KELLY,* ASSOC. M. AM. SOC. C. E.—This paper is a valuable addition to our records. The speaker, being in constant touch with the Taxing Department of Jersey City, has become familiar with the question of "land valuation", and when he read this paper he realized that it was practically flawless. In order to be sure, however, he handed it to the two men whom he thought best qualified to judge: Mr. J. Stewart Walker, formerly Chief Engineer of the Jersey City Map Department, and Mr. Raymond C. Buckley, the present Chief.

Jersey City issued last year a manual for the guidance of the working force in the Bureau of Tax Assessment of the Department of Revenue and Finance. This "Assessors' Manual", the first issued by any city in the United States, was written by these gentlemen, under the direction of Commissioner George F. Brensinger, and after reading Mr. Jerrard's paper, they also agreed that he has left little to discuss. Nevertheless, the following few points may be well taken.

If one examines cases of tax appeal one will find that the real estate man must have his builder, his architect, to support him. In the more important cases, such as railroad cases, it is the engineer who knows real estate who is called on to testify. In a town the speaker has in mind he knows of but two real estate men who have carefully compiled their sales and put them in proper form for immediate use in appeal cases.

This only emphasizes Mr. Jerrard's contention that the engineer should check the real estate man, and verifies the statement that the average real estate dealer's appraisal is but a look, a thought, an interview, and an opinion.

The author states:

"The ground rent is the net income which remains after paying the taxes on the land and the taxes, depreciation, interest, and maintenance on all improvements."

To this list of deductions there should be added the item "a sinking-fund charge sufficient, when put at interest, to cover the cost of the building in a given number of years".

How many real estate men include this item for their clients when presenting cases?

The author states that "the true value of land is the ground rent capitalized".

First: True value is spoken of by Judge Charles C. Black, in his book on "Taxation" in which he reviews many judicial decisions, thus:

"True value, like fraud, is incapable of definition, for the reason that no definition can have universal application to all the many classes of property."

He concludes with the statement:

* Jersey City, N. J.

Mr. Kelly.

"So the 'true value' of property, under the constitution for taxation, comes back in all cases to what such property will exchange for in the open market, in money. Sometimes cost is a fair criterion; earning capacity may be; frequently both should have weight; while, again, neither will determine its 'true value'; but all these elements of value should be considered."

Second: If one wishes to calculate the value of land from an income basis, one must select adequately improved property. Then if one adds to the present worth of all future anticipated net rentals of a new building, the present worth of the land value at the date when the building will need to be replaced, and deduct from the sum the cost of the building, it should closely approximate the present true value of the site.*

However, as new buildings exactly adapted to their location are rarely found, and the difficulties with this method are considerable, the sales method is to be preferred to the commercial method of finding the value.

Rarely is there an appraisal of property in which a true capitalization method is used.

In the report of the Morris Canal Investigation Committee, appointed by the joint Legislature of New Jersey, Louis Fecht, Assoc. M. Am. Soc. C. E., Engineer of State Board of Taxation, uses such a method.

He projects a possible development, assuming 2 050 ft. of bulkhead, two piers 150 ft. wide, about 2 060 lin. ft. in all. On each pier will be placed one-story steel sheds 1 175 by 113 ft. and 1 250 by 140 ft., respectively; also a six-story warehouse covering an area of 83 000 sq. ft., together with the necessary tracks, filling back of bulkhead, dredging of slips, etc.

"The commercial value has been based on the capitalization of the net surplus after deducting the probable annual fixed charges from the probable annual income.

"The annual fixed charges are made up of the interest on the investment, an annuity sufficient to create a sinking fund for the purpose of extinguishing the debt against the perishable portion of the property at the termination of its probable life; taxes on the property, annual charge for dredging slips to maintain a sufficient depth of water; insurance on the portion of the property likely to be destroyed by fire; current annual repairs; and superintendence.

"These elements will now be considered in their order:

"First: In arriving at the value of the probable annual income, rentals of dock property in the harbor have been considered. They are lower on the New Jersey than on the New York side, and range about as follows:

New Jersey side of harbor.....	$0.32 to $0.40 per sq. ft.
New York side of harbor.....	0.20 to 1.54 per sq. ft.
Average.....	0.87 per sq. ft.

* King on "Valuation of Urban Property."

"Brooklyn rental, private piers, range from \$0.48 to \$0.57 per sq. ft. Warehousing rates range from about \$0.25 to \$0.30 per sq. ft. of floor surface. Mr. Kelly.

"In estimating the probable income, a rate of \$0.50 per sq. ft. has been assumed for dock property, and \$0.25 per sq. ft. of floor surface for the warehouse. On this basis, the following result is obtained:

PROBABLE ANNUAL INCOME.

Docking, 309 000 sq. ft. at \$0.50 per sq. ft.	\$154 500
Warehouse floor area, 498 000 sq. ft. at \$0.25 per sq. ft.	124 500
"Total annual income	\$279 000

"Other minor sources of income might be added, but, for the sake of conservatism, they have not been considered.

"The rate of \$0.50 per sq. ft. is higher than the average New Jersey rentals, the development being of a higher character than other leased properties on the New Jersey side.

"An estimate of the probable cost of the development has been made, in order to arrive at the fixed charges.

"The estimated cost of the entire development for the 'Small Basin' is \$2 265 000.

"In determining the annuity to be set aside for the sinking fund, an assumed life had to be given to each depreciable portion of the plant. These values will vary with the character of the structure. Considering an interest rate of 3½% on the sinking fund, the annuities in Table 3 are obtained.

TABLE 3.

	Life.	Per thousand.	On:	
Bulkhead	50 years	\$7.68	\$153 750	\$1 173
Piers	40 "	11.83	618 000	7 311
Sheds on piers	40 "	11.83	400 000	4 732
Warehouse	50 "	7.68	755 800	5 763
Paving	15 "	51.82	9 750	505
Gantry cranes	15 "	51.82	30 000	1 555
Tracks	15 "	51.82	37 811	1 959
Totals			\$2 004 611	\$22 998

TABLE 4.—ANNUAL CHARGES.

Interest on \$2 265 000 at 5%	\$113 250
Annuity for sinking fund on \$2 004 611, at 3½% (average)	22 998
Taxes on \$1 698 750 at \$2.00 per hundred	33 975
Annual charge for dredging slips to maintain constant depth of water, 20 000 cu. yd. at \$0.20	4 000
Insurance on \$1 800 000 of dock property, at 0.5 of 1%	9 000
Current annual repairs	15 000
Superintendence	5 000
Total fixed annual charges	\$203 223

Mr. Kelly.

"In the computation of the annuities in Table 3, it will be noticed that there is no item for grading or other permanent work, which, when once completed, would last indefinitely.

"On the foregoing basis, therefore, the statement in Table 4 will represent the entire annual charges.

"With the fixed annual charges shown in Table 4, the net surplus will be:

Total annual income	$279 000
Annual fixed charges	203 223
Balance, or annual net surplus	$75 777

"Therefore, if the present worth of this net surplus, namely, $75 777, is computed on the basis of 5% interest for a term of 60 years, or the limit of the period when the canal will revert to the State, a value of $1 434 459 is obtained for the land."

The author (page 585) states as follows: "advantages of location govern the values entirely". The last word of this statement is open to question. The "Assessors Manual" quotes Mr. Alfred D. Bernard of the U. S. Fidelity and Guaranty Company, thus:

"We therefore announce four cardinal factors of value in real estate:

"1—Location, which includes access;
2—Utility, which includes capacity to produce;
3—Shape;
4—Size."

These are explained as follows:

"(1) Location.—In any city the most valuable corner lot would appear to be the lot in that city accessible to the most people.

"Now, if that proposition is sound, the most valuable corner lot is the center of population of any given city.

"In the City of Baltimore, the center of population is in the colored belt, at a point where land values are around fifty cents a square foot; so, in fixing the most valuable corner lot we must assume something else besides accessibility of location; we must assume the second factor, utility.

"(2) The highest utility which land may assume, generally speaking, is retail business, and the student of values will find that, with the exception of the banking district of New York City, the highest valued properties in all the cities may be found in the retail business districts.

"Now, with Factors 1 and 2 to resolve, we find the most valuable corner lot in any city is that lot which is so located as to be accessible to the greatest number of people who buy goods.

"After fixing the point, we must then consider the reason why one lot is more valuable than another, which calls upon us to invoke the other principles, of shape and size. The most desirably located corner may be too small to meet the business demand, and the business be carried to the next block, or, by reason of angles and irregularities,

it might not be available for the business of the territory, and thus lose in contrast with other regular lots." Mr. Kelly.

From the foregoing it is evident that the greatest number of people necessarily do not pass through the heart of the city, as Mr. Jerrard states, but more generally the greatest number of people pass near some railroad terminal or boat landing.

The retail section moves in the direction of the best residence section. Bernard sees it as follows:

"To-day the most valuable block in a given city is Main Street between X and Y Streets. Thomas Tenant's lease is about to expire. He has made big money in the twenty years he has engaged in the dry goods business in this block. His landlord wants to share his business with him and doubles his rent. Tom has made money and saved it, and feels independent enough to tell his landlord to go where ice is popular but scarce; but the landlord doesn't melt, and Tom has to pay double the rent or move. The district broker whom Tom employs to buy the site is met with 'Not for Sale', and tells Tom about four little shacks he can buy in the second block north of him and build his own building and laugh at the landlord. He does it. He builds a fine warehouse, better than the one he was in, advertises to the trade that he was forced out of his old location, and thus wins sympathy and trade. He is followed by another unfortunate, and in three years the best block on Main Street is Tom Tenant's block."

Mr. Jerrard states that the total land value of a city depends primarily on the population, but is influenced by wealth.

Bernard says:

"While the proposition may be new and startling, the writer's study of land values in cities warrants him in asserting that land values bear a relation to the number of and buying power of the inhabitants of a given community, and that in the average normal city the value of the retail business property is one cent per foot front per person and the best residence property 1/10 to 1/12 of a cent per foot per person."

In answer to the disadvantage of using gross rentals, Bernard uses such rentals, and gives, as the advantage, the fact that it eliminates the item of poor management, which invariably brings down the net rental.

If the value of the improvement is 9 times the value of the site, the gross rental should be 19% of the total investment.

8 times the value of the site	18%
7 " " " " " "	17%
6 " " " " " "	16%
5 " " " " " "	15%
4 " " " " " "	14%
3 " " " " " "	13%
2 " " " " " "	12%

Mr. Kelly.

Improvement and site equal (for office building only) 10 to 12 per cent.

To Mr. W. A. Somers is attributed the statement that the corner influence in all property generally extends 100 ft. each way from the corner. The literature of the Manufacturers Appraisal Company contained this statement, but that company acknowledged this to be an error. The speaker thinks it is an exceptional case where corner influence extends more than 25 ft. beyond the corner.

The value of lots of various depths, expressed as percentages of unit value, or "rules for long and short lots", as it is termed, requires the attention of a mathematician.

The speaker believes that Mr. Davies is the only man who gives a formula with his rule, and states that the curve is a parabola.

Interpolation, to determine intermediate values, is not satisfactory where many properties are handled. In the Jersey City Tax Office, to facilitate matters, the engineers in the Map Department have an equation for the Hoffman rule, which is in the Tax Manual published by the Department of Revenue and Finance of that city.

It is stated that Mr. Davies examined 10 000 sales. Mr. Bernard, Mr. Somers, and all the others examined many sales in order to derive their curves of values.

There is a question in the speaker's mind whether a man can be so familiar with every one of 10 000 sales as to be able to squeeze out all the influences that would pull them away from normal. Probably the law of averages would counteract errors in so great a number of sales.

The problem of depth and value presents an example resulting in an equation the curve of which, when plotted, is that of a conic section. The Hoffman rule works out as a parabola.

Knowing these results, why is it not as good to assume a condition, as Hoffman did, and work out the equation; or why not take the least number of sales or points that will determine a conic curve, and if the curve suits our judgment, adopt it.

If all the values in a curve do not respond to some formula, the speaker claims that the curve is theoretically incorrect, as no two persons are treated according to the same law.

Would it surprise the author to know that Hoffman's opinion, in 1870, agrees with Bernard's careful investigation, in 1913? If Mr Jerrard's conversion of the Bernard 150-ft. to 100-ft. rule is correc, such is the case. Unfortunately, Hoffman's complete figures are not given in Table 2. In Table 5 is given a comparison of Hoffman's and Bernard's figures. In no case is there a difference greater than 1 per cent.

In the author's comparison of the rules, it will be noticed that, up to 100 ft., the Newark business is the same as the Somers. The

Newark residence, up to 100 ft., is practically the same as the Hoffman. There is a close resemblance, up to 100 ft., of Milwaukee business and Somers, and for more than 100 ft. they agree. For more than 100 ft. Newark business and residence agree. Mr. Kelly.

TABLE 5.

Depth of lot, in feet.	Hoffman.	Bernard.
	Percentage.	Percentage.
5	....	10
10	16	17
15	23.5	24
20	30	31
25	37.5	37
30	44	43
35	50	50
40	56	56
45	61.5	61
50	67	66
55	71.5	72
60	76	76
65	80	81
70	84	84
75	87.5	87
80	91	91
85	93.5	93
90	96	95
95	98	98
100	100	100

Such peculiarities would lead one to suppose that some of the rules represent no independent investigations.

In Table 6 the speaker examines the Hoffman rule by subtracting the value of every 10 ft.—ten lower from ten higher—thus 10 ft. = 25%; 20 ft. = 41%; 41 — 25 = 16, etc. This gives a series of percentages or first differences.

TABLE 6.—Examination of the Hoffman Rule.

Feet.	Percentage.	First differences.	Subtract.	Second differences.
10	16			
		15	10	5
20	31			
		13	10	3
30	44			
		12	10	2
40	56			
		11	10	1
50	67			
		9	10	— 1
60	76			
		8	10	— 2
70	84			
		7	10	— 3
80	91			
		5	10	— 5
90	96			
		4	10	— 6
100	100			

Mr. Kelly. If 10 is subtracted from each of these we get a series of second differences in which it will be noted that 4 is lacking.

The original Hoffman rule does not plot as a smooth curve, but if 4 is supplied in this last series of numbers the following formula may be derived.

Assuming that 50 ft. gives 66⅔%, the formula is:

$$y = \frac{2x}{3} - \frac{x}{150}.$$

If 50 ft. is assumed to give 67%, the formula is:

$$y = 1.68x - 0.0068x,$$

which gives a true parabolic curve.

These formulas were derived by Mr. Walker. This peculiar condition leads to the belief that some real estate men do not understand these rules thoroughly; for, in important cases, where large tracts of land are involved, we find them analyzed by real estate men thus:

Zone 1, so many feet deep, parallel with the street, is worth a dollars. Zone 2, directly behind it, is worth b dollars, etc.

But Zone 1 reflects its value to Zone 2, etc., and immediately the straight-line valuation begins to bend, and, the smaller the zones are divided, the greater becomes the reflected increments, until the straight-line valuation becomes some kind of a curve.

The speaker believes that these rules for short and long lots are the final result of this line of reasoning, and the foregoing examination of the Hoffman rule proves it.

The appraisal of lands in different localities and sections requires individual attention and study, and for these no rules can be laid down which will fit all cases. Mr. Jerrard's paper gives a firm foundation on which to build.

Mr. Rankin. Edward S. Rankin,* M. Am. Soc. C. E.—This discussion refers to the following statement in the paper:

"The total land value of a city depends primarily on the population, but is influenced to so great an extent by the city's wealth and other factors that comparisons between cities of the same population, as to either the total land value or the value of highest priced land, offer considerable variation. It is believed that the range of values in Fig. 2 will cover the conditions in all except a very small percentage of cities."

In an attempt to formulate the relation between the population and land values in cities, the speaker some years ago addressed letters to real estate dealers in the 109 cities of the United States having a population of 50 000 or more, asking the value per front foot of

* Newark, N. J.

the highest priced land in their respective cities, based on a standard lot 25 by 100 ft., exclusive of corners. Replies were received from 47 cities, the smallest being Canton, Ohio, with 50 200, and the largest, St. Louis, with 687 000 population (census of 1910). Mr. Rankin.

The results obtained, though of a negative character, may prove of some interest in connection with this paper. They have been plotted on a diagram, Fig. 5, similar to Fig. 2, and seem to show the hopelessness of attempting to base valuations on population, except in the most general way. It would appear, however, that the curves in Fig. 5 are entirely too low.

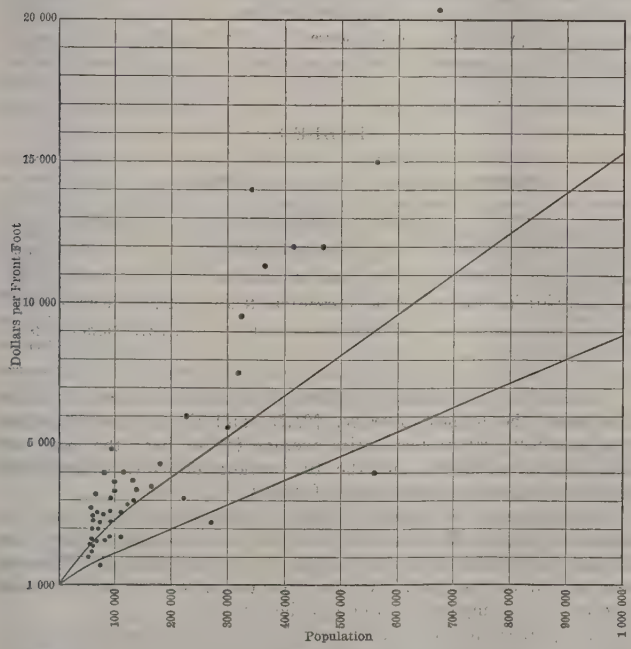


FIG. 5.

Reduced to dollars per front foot per 1 000 population, the variations are perhaps even more apparent. Some of the main features of the investigation, reduced to this basis, are shown in Tables 7 and 8.

Believing that the rate of growth of the city would have a decided bearing on the valuations—on the principle that values would discount the future—the speaker then made an estimate of the probable popu-

Mr. Rankin.

lation of each of the cities in 1920, and calculated the value per front foot per 1 000 of this estimated population. The results are given in Table 8, and show slightly less variation, but are still too far apart to be of any practical value.

TABLE 7.

Maximum.		Minimum.		Average.
All cities	$54.00	(Baltimore, Md.)	$7.16	$27.86 47 cities.
Population 50 000 to 100 000 (Springfield, Mass.)	54.00	(Somerville, Mass.)	9.71	$30.85 24 cities.
Population 100 000 to 200 000 (Grand Rapids, Mich.)	35.52	(Nashville, Tenn.)	15.80	$26.64 10 cities.
Population 200 000 to 400 000 (Newark, N. J.)	40.28	(Jersey City, N. J.)	8.06	$23.12 8 cities.
Population more than 400 000 (St. Louis, Mo.)	29.48	(Baltimore, Md.)	7.16	$23.59 5 cities.

TABLE 8.

Maximum.		Minimum.		Average.
All cities	$38.40	(Jersey City, N. J.)	$6.29	$20.33 47 cities.
Population 50 000 to 100 000 (Springfield, Mass.)	38.40	(Somerville, Mass.)	7.65	$21.33 24 cities.
Population 100 000 to 200 000 (Albany, N. Y.)	31.13	(Birmingham, Ala.)	9.00	$20.29 10 cities.
Population 200 000 to 400 000 (Newark, N. J.)	28.86	(Jersey City, N. J.)	6.29	$18.77 8 cities.
Population more than 400 000 (St. Louis, Mo.)	25.00	(Baltimore, Md.)	6.78	$18.10 5 cities.

Classified as to location, into East, West, and South, the 17 eastern cities show the highest average, $21.75, and the greatest variation, $6.29 to $38.40; 20 western cities average $21.45, with the least variation, $11.84 to $37.62; and the 10 southern cities vary from $6.78 to $23.69, with the lowest average of $15.71.

All the populations used in Table 7 were taken from the U. S. Census for 1910, and do not include suburbs tributary to the cities, although the speaker fully agrees with Mr. Jerrard that such suburban areas should be included. At the time the investigation was made, however, it was impossible to obtain these figures.

The speaker would suggest further, that an important factor in the value of land is the volume of traffic passing the point in question. A careful investigation along this line might prove of considerable interest.

Mr. Thomson.

T. Kennard Thomson,* M. Am. Soc. C. E.—Two interesting examples of the valuation of land in New York City, might be cited: First, the corner of Wall and Nassau Streets; a lot, about 25 by 75 ft., was bought

* New York City.

in 1896 for $500,000, and, after the old building had been torn down, a 17-story building was erected on the site at a cost of $1 000 000, making the cost of the new building and lot $1 500 000. The Bankers Trust Company, 14 years later, wanted the corner to complete its property in order to have about 90 ft. front on each street, so the 17-story building and lot was bought for $1 500 000 and then $50 000 was paid to a wrecker to remove the building quickly. Mr. Thomson.

The second case is that of a young man who had an assured income of $25 000 a year; he painted a little, wrote a little, and tried to make himself generally useful, and at the same time was able to live comfortably on his income. An uncle bequeathed him a $1 000 000 piece of property on West Street. Just then, unfortunately, the tenant moved out, and a new one could not be found, and as the property was subject to a mortgage which required nearly all the young man's $25 000 income to pay, he was absolutely impoverished by the bequest of a $1 000 000 piece of real estate, which he could neither rent nor sell.

One of the great stumbling blocks in modern real estate is the old 25-ft. front lot, which was all right in the days of one- or two-story cottages and stores, but is absolutely out of place in a modern city, and is responsible for much of our "crazy quilt work". It is to be hoped that future cities will be laid out on up-to-date lines, giving a whole block for each building, then the question of the relative values of the inside and outside of the block will disappear.

An interesting case occurred some years ago, when the caisson foundations of a new building encroached on the adjoining property at depths of from 30 to 90 ft. below the surface.

Mr. Jerrard's admirable paper has one defect which should be remedied before the final publication, thereby obliterating this comment thereon. It is the impression received by the reader that the engineer is a better authority on real estate than the real estate man. No engineer would grant that a real estate man, pure and simple, could do better engineering than the engineer. It should be an easy matter to remove this slight flaw from the paper, and the speaker trusts that the author will make this correction.

J. S. Walker,* Esq.—The speaker, until lately, has been connected with the Tax Department of Jersey City, and in his position witnessed many contests between the City and the corporations. In connection therewith he has heard able men discuss the various phases of this question. He realizes the fact that this paper represents a great deal of study, is to the point, and is correct. Mr. Walker.

The most interesting feature is the theory of the value of the fractional part of a lot. As Mr. Kelly has stated, Hoffman's opinion,

* Jersey City, N. J.

Mr. Walker.

in 1870, was that the front half of a lot was worth two-thirds of its total value. Hoffman produces a series of numbers; as to how he derived them we are not told, but they happen to be a series representing a conic section.

The conversion of Mr. Bernard's 150-ft. rule into the 100-ft. rule by Mr. Jerrard, agrees with the Hoffman Rule, which the speaker thinks is very peculiar.

The little investigating the speaker has done tends to show that the problem of the relation of depth to value results in an equation of the second degree, between two variables, and is an equation of a conic section.

In practice a curve is constructed or adopted, and, under certain conditions if it gives too much or not enough value, some other curve is copied in whole or in part. The curve then varies in such a manner that it cannot be represented by a formula. Such being the case, no two conditions are treated according to the same law.

A case in point is that of a district in transition from one use to another, such as apartments taking the place of private dwellings, the appraiser claiming that more value is found to a greater depth for land on which was built apartments than that devoted to private dwellings, and a variation of the curve in use was applied.

In 1912, or about that time, Newark had curves which differed from those in use at present, to which the foregoing objections would apply. The subject calls for extended and exact study.

The Manufacturers Appraisal Company, of Cleveland, Ohio, which uses the "Somers Unit System of Realty Valuation", does not reveal the method of calculating the value of corner lots, irregular lots, or reflected influences. It appears that a city hires only the use of the system, and, should it be desired to re-adjust values, it would be necessary to re-hire the system. Nevertheless, all things about the Somers System are not kept secret, and it has many good features.

First of all, it is a system that is carefully worked out. The speaker thinks it overcomes some of the objections referred to by one of the speakers.

A booklet, published by the City of Cleveland, Ohio, entitled "The First Quadrennial Assessment, Cleveland, Ohio", shows how Cleveland was assessed by the Somers System in 1910, and gives many details and charts.

Mr. W. I. King,* appears to have gotten nearer to the heart of the system than any one else, and shows some of the Somers charts not found elsewhere, but he says:

"Since the scientific accuracy of the Somers tables is not proven, and since these tables and their derivation are shrouded in secrecy,

* *Bulletin No. 689* of the University of Wisconsin entitled "The Assessment of Urban Property."

it seems necessary to derive a method of corner valuation, and ultimately a series of charts, which may lay some claim to a scientific basis, and at the same time, accord with actual conditions." Mr. Walker.

This bulletin is of value to those interested in taxation.

In assessment work, Mr. Somers lays much stress on the value of community opinion. He writes as follows with reference to an assessment in Des Moines, Iowa:*

"The first public meeting was held; * * * about seventy-five prominent citizens were present, including a majority of the largest property owners. After the session a committee of citizens and property owners was selected by the Mayor and Assessor to co-operate with, and assess the latter.

"At the outset it was decided to avoid discussion of actual values at first and to deal with relative values only. Therefore the efforts of the Committee were devoted to expressing in percentage the relative values of a Somers unit foot on each side of every block.

"It was the unanimous opinion that the most valuable frontage in the city was on the north side of West Walnut Street between Sixth and Seventh Streets. This the Committee called x, or 100 per cent. Then, at the several meetings, the relative value of a unit foot in every block throughout the West Side was ascertained.

"When the relative values were compiled, a map of the district was prepared, and the percentages placed thereon. Copies of this map were distributed and printed in the newspapers; another meeting was called, and criticism of the relative values invited."

Another section of the city was likewise treated.

"When the two committees had completed their labors, it remained only for the assessor to determine x, or the 100%, on West Des Moines Street." The value of x, or 100%, also came under discussion. "Calculating clerks were then employed and trained in the methods of the Somers System. As soon as the unit values were definitely ascertained, the work of calculating began."

The opinion of the taxing authorities throughout the United States seems to be that the "Newark System" is fundamentally the best, but it has taken 20 years to build it up. In Jersey City, a system is being constructed by testing any idea that looks good in any system, and is likely to develop into one which will be the best in the country.

William J. Boucher,† Assoc. M. Am. Soc. C. E.—The speaker desires to present a few thoughts regarding the peculiarities of certain sections in cities which seem to have a direct bearing on valuation. Mr. Boucher.

One of the busiest retail business streets in Manhattan (New York City) is 42d Street, and every day, including Sundays, its sidewalks are filled with pedestrians and the roadway with cars and vehicles. Between Fourth and Eighth Avenues, during the past 10 years, there

* As published in the June, 1913, issue of the *Somers System News*.

† Long Island City, N. Y.

Mr. Boucher.

have been erected several high-grade office buildings, one first-class department store, numerous smaller stores, some excellent hotels and restaurants, several banks, the Central Public Library, and the New York Central Terminal Station. All these tend to make this one of the most popular and attractive streets in the city, and the rentals,

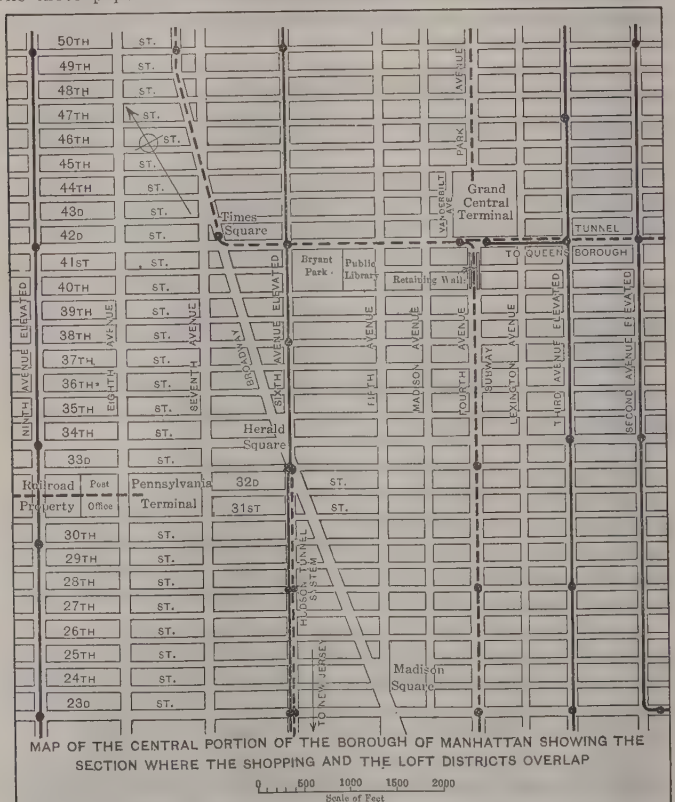


FIG. 6.

and consequently the valuations, are high. Contrast it with 41st Street (Fig. 6), only 200 ft. away and parallel with it. True, it is not a through street, but though this affects vehicular traffic, it is strange that the street contains so little to attract the pedestrian. In the 400 ft. between Madison and Fifth Avenues, there is only one

high-grade office building. This has a frontage of 50 ft., is 100 ft. deep, and 20 stories high. It enjoys close contact with the Rapid Transit Subway, as one has merely to cross the street and enter the building on the opposite side, which has a subway entrance on its 42d Street front. The remainder of the block is occupied with small shops, restaurants for light, noon-day refreshments, and other establishments paying small rents. Mr. Boucher.

Again, consider 43d Street, which is 200 ft. to the north of 42d Street. It is a thoroughfare from the Grand Central Station to the Hudson River, but it has practically no retail, or other business to make large rentals or valuations. It is given over largely to clubs and other buildings where gatherings can be accommodated. One office building fronting on 42d Street, with a subway entrance in front of its door, has an "Annex" building on 43d Street. The buildings are connected by a through corridor giving the "Annex" the full privileges and benefits of the 42d Street entrance, but the rentals are considerably less than those for the 42d Street front. The contrasts of these two contiguous streets (41st and 43d), so close to 42d Street, form an interesting study.

Reference is made in the paper to the advantages of transportation, and it does seem as though that is the originating feature of value of land, although the speaker makes no claim to being an expert. One location stands out prominently, *viz.*, 149th Street and Third Avenue, and vicinity, in the Borough of the Bronx, which is a goodly distance from "down town", yet, through the influence of good transit facilities (both "Subway" and "Elevated", as well as surface cars), there has grown up in that section in the 12 years since the subway was opened to traffic, a very busy retail business center, where rentals must be quite high. Comparing this location, about 5½ miles from 42d Street, with its good transit facilities, with another location only about 1½ miles from the Grand Central Station, which has until recently had only the poorest, or practically no, transit, and the contrast is remarkable. This other location is in the Borough of Queens, directly across from 42d Street. It is true, it is separated from Manhattan by the East River and, until recently, that stream formed a most effectual barrier to progress. Two ferries, one at 34th and another at 92d Street, gave occasional access across, and brave was the pioneer who had his home or his business there. About 10 years ago, surface car service was established over the Queensboro Bridge from 59th Street, Manhattan, to Queens, but not until the Steinway or Queensboro Subway was opened, in June, 1915, with its free transfer privilege to the main subway line, could it be said that rapid transit existed there.

On the river front opposite Manhattan, from 45th to 59th Streets, there still remain large tracts suitable for wharves and warehouses,

Mr. Boucher.

and, farther inland, there are other large tracts, with railroad facilities, and now served with rapid transit, simply waiting to be developed with industrial establishments of every kind, with desirable locations for workingmen's homes near-by. These tracts may be had on very favorable terms, and the valuations are low when compared with other localities much farther removed from the business centers of Manhattan.

It is very unfortunate that real estate in the Borough of Manhattan, which, for so many years, has been considered a safe and desirable investment, should have proved so unstable and shifty in the past 10 or 15 years. Since the advent of the modern, steel skeleton building, with its cheap curtain walls and concrete floors, it has offered peculiar inducements for the manufacture of garments of white goods and clothing, in which lines large numbers of men and women are employed. This line of manufacturing, until quite recently, was conducted in the district south of 14th Street, in buildings of the old type so common all through that section. About 10 years ago, many progressive, but unfortunately, as it proved, misguided owners or speculators seized upon the section north of 14th Street, and, in practically every east and west street, erected many of the so-called "modern loft" buildings of ten and twelve stories, in which these garment trades are housed. The thousands of operatives are mainly recruited from the recently arrived immigrants, and, during the noon hour and at opening and closing hours, the sidewalks and streets literally are covered with the slow-moving, jostling, noisy swarms. At the time this movement of these trades was taking place, an entirely different movement was under way. The department stores which, 15 years ago, were on 14th Street and on Sixth Avenue, north of 14th Street, and on 23d Street, began to move to newer and more commodious quarters in the vicinity of 33d and 34th Streets, Sixth Avenue, and Broadway, and a little later to Fifth Avenue, north from 34th Street. Thus, the new "loft" district and department store district overlapped and conflicted to such an extent that the very existence of the department stores was threatened by the swarms of noon-day loiterers from the "lofts", who slowly wandered up and down and along, completely blocking the passage of would-be patrons of the stores.

The nuisance was so great that the "Save New York" movement has become a reality, and is already bearing fruit. The first result was the legal zoning of the city, during 1916, by which the character of various districts will be restricted, and in the different "zones" certain restrictions will apply, which will prevent the intrusion of businesses or trades into districts which would have their real estate values impaired or ruined by such intrusion. The "Save New York" move-

ment is considerably different from the zoning plan in that it involves a drastic remedy for existing evils in the district bounded by 32d Street (north side) to 59th Street, and from Third to Seventh Avenue. In brief, the scheme of the Committee was: To persuade manufacturers who contemplated moving into this zone to stay out; to induce those manufacturers already in, to move out; and to stop the erection of new manufacturing "lofts" in the zone bounded as described. An agreement was also made by buyers, to give the preference in buying goods to those concerns who do not manufacture in this zone, this measure to take effect on February 1st, 1917. The response to the Committee's appeal was immediate, and up to January 1st, 1917, the following results have been accomplished: First, not a single new lease for manufacturing purposes has been made in the zone; second, not a single new building for manufacturing purposes has been erected in the zone; and third, all but 20 firms, out of a total of 225 now manufacturing in the zone, have agreed to move out at the expiration of their leases, at various times during 1917, 1918, and 1919. Mr. Boucher.

Thus the future of the district is assured by the new "zoning" plan, although tenants of a different class must be found for the buildings already erected which will be vacated.

Referring again to the comment that it is cheap, convenient, and frequent transit which apparently forms the basis of land values in the first instance, there are many persons who well remember when the districts lying north of 59th Street, Manhattan, were sparsely settled, where rocky hillsides and trees abounded, and where goats roamed at will in and around the shanties of the "squatters" who were the first settlers. At that time the means of transit were confined to horse cars and omnibuses, and as the time required to travel from the business sections at or around the City Hall to the neighborhood of 59th Street was fully one hour, it was about all that the patience or the physical health of the passengers could endure, as the cars were not heated and the feet found cold comfort in the straw loosely placed on the floors in winter. The elevated railway system was opened for traffic before 1880, and by 1884 had reached Harlem, and with its advent came the buildings and the people to fill them. A later instance, within the memory of all, is the Washington Heights Section of Manhattan and the sections of the Bronx made readily accessible by the subway opened in 1904. Until that date, only surface cars reached those districts, but, during the first 5 years after the trains began running, entire neighborhoods were transformed from open vistas and low valuations to closely built blocks of buildings with much enhanced values.

Again, this matter of the influence of transit facilities is very noticeable in Chicago. The four elevated railroad main lines are the

Mr. Boucher.

Northwestern serving the "North Side" and northwest section, the Oak Park (Lake Street) and Metropolitan serving different sections of the "West Side" and the South Side line serving the southern and southwest sections. Each has a very heavy morning and evening rush-hour traffic. The four lines reach the business section, as shown on the map, Fig. 7, and there jointly use the "Union Loop" on Wabash Avenue, Lake Street, Fifth Avenue, and Van Buren Street. The "Loop" was opened for traffic in 1897 and, until recently, all trains entering it passed entirely around it and left it at the point of entrance. It was merely a joint terminal, and fares began and terminated there; if a passenger desired to continue his ride, he left one train, went into and through a station, paying another fare and boarded the train of another company also stopping at that station platform. During the past 3 years, however, "through routing" has gone into effect, by which trains are run through from the North to the South Side and *vice versa,* and free transfers are issued from both these lines to the "West Side" lines.

The "Loop" has been termed both a blessing and a curse. In the first instance it provided a splendid terminal arrangement for all trains, from whatever section, giving preference to none and permitting passengers the maximum of convenience in the matter of reaching the down-town section, but, on the other hand, it has undeniably had the effect of creating the most congested business section in the United States. From Wabash to Fifth Avenues is about 2 000 ft., and between Lake Street and Van Buren Street is approximately 3 100 ft., and within this area of less than ½ sq. mile is concentrated the business center of America's second city. Towering buildings rise on every street, each street is almost equally thronged, although State Street, with its department stores, is noticeably the busiest. Pedestrian, car, and vehicle traffic combine to produce the very limit of impeded traffic. By a reliable count, made by unprejudiced observers in 1910, the number of pedestrians on State Street, passing the corner of Madison Street, was so great that, without doubt, it might be termed the most densely traversed locality in the world. Any fair Saturday afternoon, the 20-ft. sidewalk is thronged with densely-packed, slowly-moving crowds, which fill it from curb to building line, about equally divided between those moving north and those moving south, on both sides of the street.

In this section are the large retail stores, many wholesale stores, some warehouses, the banks, exchanges, hotels, theaters, office buildings, some light manufacturing establishments, in fact, everything that tends to draw people; the result has been that every foot of ground has been utilized by the erection of business buildings, mainly of a very good type, with the corresponding result that valuations and rentals are high.

Step beyond the limits of that "Loop", and on all sides (the East Side excepted) one immediately notices the difference. The buildings are of a different character, and the hurrying crowds are absent. Mr. Boucher.

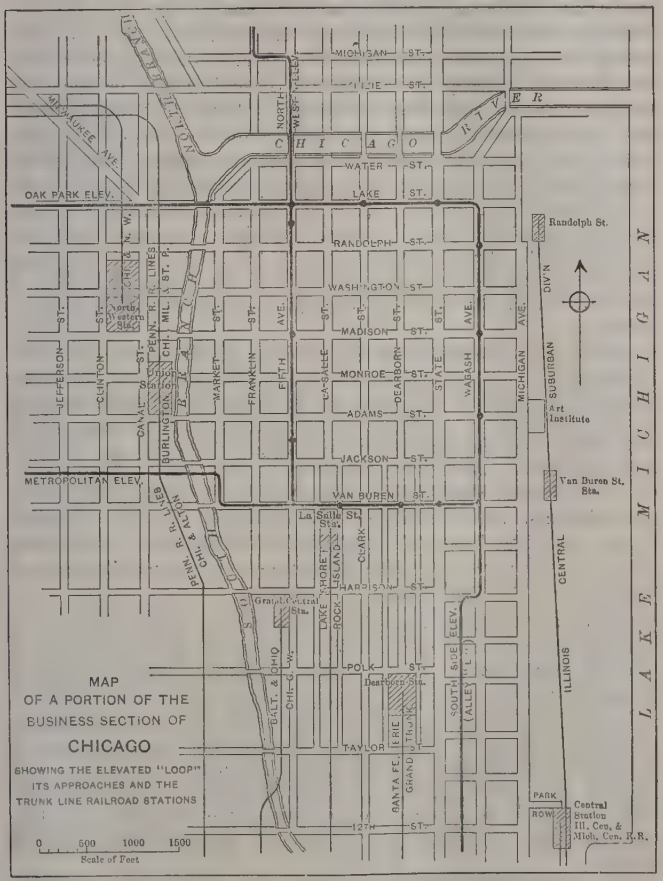


FIG. 7.

East of the Loop on Wabash Avenue, the city extends one block to Michigan Avenue, overlooking Grant Park and the Lake. Michigan Avenue, like Fifth Avenue, New York City, has been kept free of

Mr. Boucher. street cars, its traffic resembles that of the eastern city, except that it has no 'buses, and its development has been very similar. The buildings are of the highest class, consisting of office buildings, first-class jewelry and retail stores, clubs, places of musical entertainment, the Public Library and the Art Institute. Mr. Jerrard's Fig. 1 is a view looking north, as indicated on the map, Fig. 7, showing Grant Park, with Michigan Avenue on the left and Lake Michigan on the right. Marvelous, indeed, have been the changes. The Illinois Central Railroad is shown crossing the pond or arm of the lake on a wooden trestle. All the neighboring section has been filled in, so that the general level is 14 ft. above the lake, and the park has been enlarged to the extent shown on the map. The Suburban Division of the railroad traversing the park is confined within a depressed way, with retaining walls. On the left, instead of the church spires and comfortable looking dwellings surrounded with lawns, now stretches an unbroken line of high-class buildings giving high valuation to all the section.

Across the Chicago River (at the top of the map), begins the "North Side". Here, only a step, as it were, from the busy, high-value center, is a neglected, low-value section extending for approximately a mile northward. It is given over to rooming and boarding houses, with warehouses and some manufacturing. Similarly, the territory adjoining the "Loop" on the west and south sides contains large areas of low value, with cheap tenements, mixed in with manufacturing establishments, warehouses, and railroad yards, awaiting a Master Hand, with Capital and Genius to transform them into a harmonious scheme to produce the income in tax returns which such advantageously located property should bring.

Mr. Mayo. Franklin F. Mayo,* Esq.—The value of land is not constant, but is always moving up or down in the scale, with a period of stationary value, the time element of these three phases being uncertain, owing to changes in the factors producing the value. Also, certain locations and districts possess a premium value over the income value, at certain periods. The speaker contends that this premium value is uncertain, and should be considered in all taxing valuations, as a property in a district where values are declining should be taxed (that is, valued) at a price based on the income obtainable on a lease made at that time, and not on a basis of sale during the premium period.

To do this would require a keen sense of the human element factor, and no method of valuation would be satisfactory unless made by men especially qualified in placing values.

* Newark, N. J.

Mr. Jerrard.

L. P. JERRARD,* JUN. AM. SOC. C. E. (by letter).—The discussion of this paper has brought out several points which may be cleared up by further explanation. The writer's loose application of economic terms has evidently obscured his argument in several instances. Mr. King and Mr. Kelly both take exception to the statement: "The true value of land is the ground rent capitalized." Inasmuch as land has value because it is a source of ground rent, it would seem that the market value should bear some definite relation to the amount of ground rent. Such is not the case, in view of the fact that the ground rent is subject to fluctuation, and market values discount the prospective changes in the amount of ground rent. The intention was to bring out the distinction between market value and a value based entirely on ground rent capitalized, which was inaccurately designated by the term "true value". The statement, "The basis of business values is strictly economic, * * * but the basis of residence values has a social element," is also challenged by Mr. King, who states that the demand of wealthy people for pleasant residence sites is as strictly economic as the demands of any business utility for land. The social element referred to by the writer manifests itself in the demand of people of lesser wealth to live as near as possible to the wealthiest class. Of two residence sites, all other factors (such as schools and transportation facilities) being equal, the one nearest the best residence section of the community is ordinarily preferred. It would seem as though this tendency is more sentimental than economic, but, in either case, it was the distinction between business and residence values which was intended to be made.

Mr. Kelly has brought up several points to which the writer must take exception. To the deductions to be made from gross rentals in computing ground rent Mr. Kelly would add, "A sinking fund charge sufficient, when put out at interest, to cover the cost of the building in a given number of years." This charge is covered by the item "annual depreciation" which was included in the list of deductions. "Advantages of location govern the values entirely." By "values", as used in this statement, is meant values in general, or unit values, which would eliminate the factors of shape and size suggested by Mr. Kelly. He also suggests as a factor governing values, "utility", but does not utility depend entirely on location? Land has greater value when utilized for business purposes than for residence property; but it cannot be used for business purposes unless it is in the business section of a community, and even then its value depends on its particular location within the business section.

Mr. Kelly understands the writer to say that the greatest number of people pass through the heart of the city. Possibly he has in mind

* Chicago, Ill.

Mr. Jerrard.

the statement, "retail stores gather at the heart of the city" where the largest number of prospective customers will pass. It is frequently true, as Mr. Kelly states, that the greatest number of people pass some railroad terminal or ferry landing, but they are hurrying to and from their work and are not "prospective customers." It may be interesting to note that such corporations as the United Cigar Stores Company, and F. W. Woolworth Company, before locating one of their stores, make a careful investigation, not only of the number of people passing a prospective location, but of their wants, buying power, and inclination to buy at that point. It is possible that a cigar store would flourish at a terminal where no one would find time to patronize a five and ten-cent store. A traffic count might show a great number of people passing a certain corner, but, if they were mostly factory girls and women, it would not augur well for a cigar business.

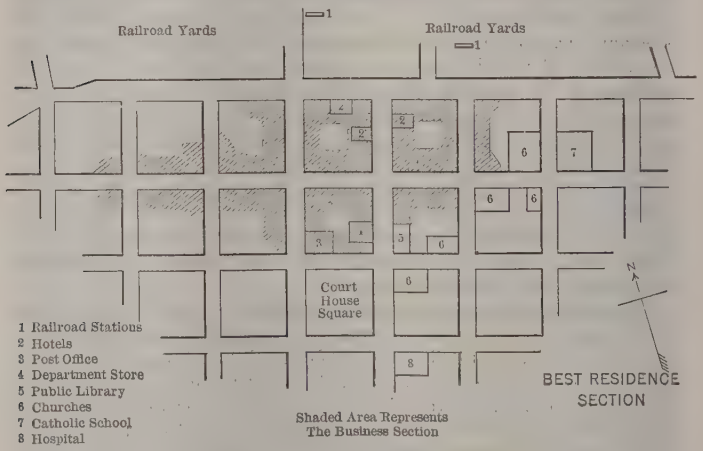


FIG. 8.

"The retail section moves in the direction of the best residence section." It is true, as Mr. Kelly suggests, that local conditions sometimes alter or retard this tendency, but the tendency persists nevertheless. The writer noted an interesting situation recently in Watertown, S. Dak. The sketch, Fig. 8, is from memory only and not accurate, but illustrates the situation approximately. The business section is separated from the best residence section by a zone of public buildings, churches, and other permanent institutions, which must form a substantial barrier to the extension of the business section in that direction. To the west and north, where there is room for the expansion

of the business section, there is nothing more attractive to business than railroad yards and river bottoms. The tendency of retail business to reach out toward the best residence section seems to be illustrated by the location of a large department store, the best in the city, on the edge of the business section, but on this side most convenient to the best residence district. Mr. Jerrard.

Mr. Walker speaks of the Somers system of assessing cities, with the aid of community opinion. In fixing the point of greatest value at 100%, and determining the value of all surrounding frontage in percentages thereof, as explained by him, it would seem that all errors which may be introduced are cumulative. If an incorrect relative value is established for any block, the error is reflected in a series of blocks the values of which are relative to the one in error. The writer has seen evidence of the partial failure of the Somers system in this respect, and has also noted that, in the public meetings and discussions, the principal efforts are made by those land owners who are most directly interested, and that the conclusions are likely to be biased in their favor.

Mr. Rankin has introduced some interesting and pertinent data with regard to the value of the best business land in cities. With respect to the discrepancy between the points plotted by Mr. Rankin and the curves advanced by the writer (Fig. 2), the following influences are suggested as regards Mr. Rankin's data:

1.—Population of cities does not include tributary suburbs (mentioned by Mr. Rankin);
2.—Undue optimism which frequently prevails among real estate dealers regarding land values;
3.—Difficulty of determining, in the business districts of a city, values which are free from corner influences.

All these factors would tend to throw the points plotted by Mr. Rankin away from the curves. Part of the data on which these curves were based, are here submitted, and also shown in Fig. 9. Mr. Alfred D. Bernard, in "Some Principles and Problems of Real Estate Valuation", states as follows:

"In a normal city for populations above 50 000 the value of the best retail business property is one cent per front foot per person.

"City.	Population.	Best Land per Front Foot.
Boston	1 500 000	$15 000.00
Philadelphia	1 500 000	15 000.00
Baltimore	650 000	6 000.00

"Baltimore has a large negro and low wage earning population and values are below normal. Cleveland, Louisville and Minneapolis will prove the rule."

Mr. Jerrard.

Mr. Richard M. Hurd* has given the figures reproduced in Table 9. From miscellaneous sources, including some of the writer's own determinations, city assessments by the Somers method, and opinions of real estate dealers, the data in Table 10 have been collected.

TABLE 9.

City.	Population.	Best business per front foot.
Washington	270 000	$5 000
Louisville	205 000	1 700
Minneapolis	203 000	2 500
Indianapolis	169 000	2 500
Kansas City	164 000	2 500
St. Paul	163 000	1 800
Denver	134 000	1 800
Toledo	132 000	2 000
Memphis	102 000	2 000
Portland, Ore	90 000	1 600
Atlanta	90 000	2 000
Richmond	85 000	1 800
Seattle	81 000	2 000
Des Moines	62 000	1 500
Salt Lake City	54 000	1 400
Duluth	53 000	1 000
Spokane	37 000	800
	25 000	$300 to $400
	50 000	600 to 1 000
	100 000	1 200 to 2 000
	150 000	1 500 to 2 500
	200 000	1 800 to 3 000
	300 000	2 500 to 4 500
	600 000	4 000 to 7 000
	1 000 000	7 000 to 10 000

TABLE 10.

City.	Population.	Value of best business per front foot.
Boston, Mass	1 000 000	$15 000
Cleveland, Ohio	580 000	5 500
Milwaukee, Wis	420 000	6 000
Indianapolis, Ind	250 000	3 000
Des Moines, Iowa	94 000	2 000
St. Paul, Minn	230 000	2 500
Minneapolis, Minn	325 000	3 500
Superior, Wis	45 000	700
Racine, Wis	40 000	600
Madison, Wis	30 000	2 000

In Mr. Thomson's opinion, this paper suggests in too strong a manner that the engineer is a better authority on real estate than the real estate man. There is no implication that the engineer has

* In "Principles of City Land Values."

any place in the general field of the real estate business, which includes platting, buying, selling, renting, and managing property, and the question pertains only to making land valuations. In valuation, it is felt that some of the work of real estate men is inaccurate, and that, in many cases, some effort should be made to check the appraisals submitted by them. This is especially the case in smaller cities. As an indication that many valuations made by real estate men are in error, the writer may cite the frequent instances of wide variations in the valuation of the same property by different appraisers. The writer has had some experience, in cities of less than 50 000 population, in obtaining two valuations of the same property from the same man at intervals of several weeks or months. It is not uncommon to find

Mr. Jerrard.

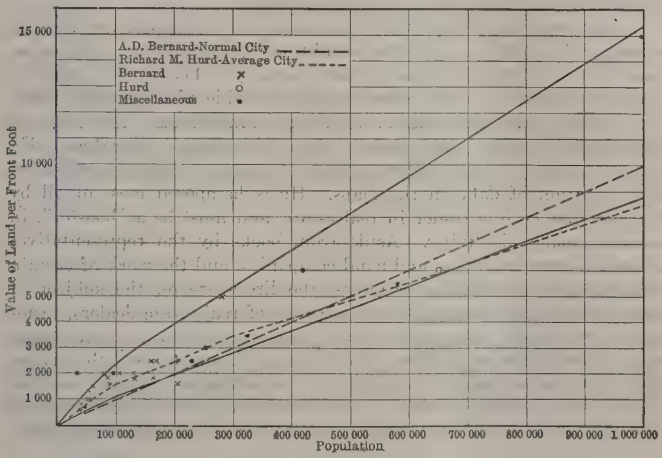


FIG. 9.

considerable variation in the results in such cases. If a deliberate attempt is made to confuse the real estate man by argument, or by introducing units of value with which he is unfamiliar, a very wide variation in results may be obtained under the same circumstances. In all these cases, an effort was made to obtain the services of the best informed man in the community.

Some organizations, which require land valuations to be made in different cities, proceed in the following manner: A complete record of real estate sales is obtained from abstractors and from public records. The services of a local real estate man are obtained, and he states what unit of value is most familiar to him, such as the lot, square foot, or front foot. The sales data are then reduced to the same unit and

Mr. Jerrard.

plotted on a map, together with information collected as to the extent of improvements on properties sold, assessment records, etc. The map is then turned over to the expert, who studies the situation and makes the valuation of the land in question. Satisfactory results are obtained in this manner. The experience of such organizations has been that, if the data are not provided for the expert, he will not obtain all of them, nor will he reduce them to the form in which they are most applicable to the problem. If the information is expressed in units with which he is unfamiliar, he cannot readily understand it or convert it to another unit. It is not unlikely, in cases such as these, that the man who collects and prepares the data becomes as familiar with the values involved as the local expert, but, in a contest, the local expert is the only one who can qualify as a witness.

A true real estate expert is a man who has had experience in land values, good judgment of actual conditions, foresight into tendencies, freedom from prejudice, and natural honesty. The field is one for a specialist, but it has not been developed outside of the largest cities. The average real estate dealer is not a trained man. He does not systematically collect and classify data with regard to values. He is not familiar with the different units of value, or with the graphical representation of data on the maps. He is hampered most of all by the fact that he is a trader in real estate, and must be a "booster", in order to make his living. Aside from books by the representatives of several large mortgage and bond companies, and the work of certain men engaged in taxation problems, the literature on the subject of land valuation is meager. In the writings of real estate dealers, terms such as "plottage", are rarely defined, and are used in a contradictory manner by different authors. Under the circumstances, the writer feels that the engineer in charge of a valuation may sometimes find occasion to investigate with his own staff the item of land, without encroaching on a field adequately covered by men of another profession.

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Paper No. 1393

TESTS OF CONCRETE SPECIMENS IN SEA WATER, AT BOSTON NAVY YARD*

By R. E. Bakenhus, M. Am. Soc. C. E.

With Discussion by Messrs. T. Kennard Thomson, J. J. Yates, J. R. McClintock, S. B. Williamson, Waldo C. Briggs, Charles S. Bilyeu, W. E. Day, Robert Ridgway, George W. Fuller, A. H. Rhett, Marshall W. Brown, Albert Larsen, W. Watters Pagon, R. J. Wig and Lewis R. Ferguson, and R. E. Bakenhus.

Synopsis.

This paper describes a test of twenty-four concrete specimens which were immersed in sea water for 7 years. The object of the tests was to determine the action of sea water on concrete specimens of wet and dry consistencies, of various proportions of ingredients, and of different brands of cement, as well as the effect of special compositions.

The methods of mixing, analyses of the various cements, sand, and stone, and the conditions of the test, as well as all other data having possible effect on the results, are stated in the paper. The information is given in tabular form where possible.

The specimens were examined at intervals of about one year, and record was made of their condition. The results of these observations have been tabulated, and show progressive deterioration of some of the specimens and remarkable durability of others. Recently, the specimens were examined with great care, and graded in the order of durability. These results are also tabulated. Independent tabulations are made of the various series of tests originally planned, to

* Presented at the meeting of January 3d, 1917.

ascertain in one case the effect of wet and dry mixture, in another case the effect of rich and lean mixture, and in others the effects of special brands of cement, and of using lime, Sylvester wash, etc., with the cement.

The results are interesting, and seem to show, briefly:

(*a*) That the 1:1:2 mixture is superior to the 1:2½:4½, and that the 1:2½:4½ is, in turn, superior to the 1:3:6;

(*b*) That the wet mixtures are superior to the dry;

(*c*) That the effects of magnesia or alumina in varying proportions are not very marked, and follow no apparent law, although the two most durable specimens are those lowest in alumina content;

(*d*) That extra care in mixing produced decidedly beneficial results;

(*e*) That hydrated lime was of no benefit, but rather a detriment;

(*f*) That the addition of Sylvester wash was harmful; and

(*g*) That the addition of clay to the cement had a slightly beneficial result.

The deterioration occurred between high and low water, and was most marked at mid-tide. Above high water there was little deterioration, and the same is true, but to a less marked extent, of the concrete continually submerged.

A careful reading of the paper should be made before any of the foregoing conclusions are applied. The experiments are not sufficiently extensive to warrant drawing final conclusions in all cases, unless confirmatory evidence is available. In utilizing the results, the limitations of the tests and local conditions should be taken into account.

About 7 years have elapsed since twenty-four concrete specimens were placed in sea water at the Boston Navy Yard. The results, on careful analysis, give information of considerable value. This paper presents a résumé of the important facts in regard to the preparation of the specimens, the recorded observations at stated periods, and a tabulation and discussion of the results to date.

The various facts and observations connected with the tests are given in full, so that those who wish may study them and draw their

own conclusions. Those who are sufficiently interested may observe the specimens at the Navy Yard. In the latter part of this paper the principal results of the tests are summarized, and the conclusions of the writer are stated, without attempted explanation, however, as to the causes of the different kinds of action on the concrete specimens.

The expense of making the tests is borne entirely by the Aberthaw Construction Company, of Boston, who proposed them and entered into agreement with the Navy Department, represented by the late Fred Thompson, M. Am. Soc. C. E., Civil Engineer, U. S. Navy. The agreement (Appendix A) covers the conditions of the tests whereby the specimens were permitted to be made and exposed in the Navy Yard, all work being under the inspection of the Civil Engineer, now Public Works Officer, of the Yard, and the Navy Department having the right to observe the specimens and to use the information obtained.

Each of the twenty-four specimens is 16 ft. long and 16 in. square, weighing from 4 050 to 4 350 lb. They are hung on Pier 9, at the Boston Navy Yard, in such a manner that the lower ends are continually immersed, the upper ends but rarely immersed, and the center portions are subject to alternate action of the air and salt water due to the 10-ft. tides.

The specimens were originally suspended, in February, 1909, from the old Pier 9, but, as this was rebuilt, they were moved on December 5th, 1911, to Pier 8, east side. Later, it became necessary to renew the supporting bridles, and, on September 19th and 20th, 1912, the specimens were removed and leaned against the quay wall and on October 30th and 31st, 1912, were moved to the newly constructed Pier 9. The piers are of open wooden pile construction, giving the sea water a free flow. About April, 1914, a pipe line for furnishing fuel oil to naval vessels was placed on Pier 9, and since then the concrete has become coated with a thin film of fuel oil. The sea water is that of Boston Harbor, analyses of which will be given. Fig. 1 is reproduced from a photograph showing the specimens as suspended from Pier 9 in April, 1916.

The specimens were lifted out of the water on December 9th and 10th, 1913, for the purpose of photographing them, and were immediately replaced.

The following description of the manufacture of the specimens is taken from the file copy of the records made at the time:

"The specimens were made on the first floor of Building No. 107 at the Navy Yard, which was kept above freezing point but yet not very warm. The feeding chute was removed, and materials were shoveled directly from the floor to the mouth of the mixer, which was about 3 ft. above. The materials were dumped into a water-tight mortar box, from which they were shoveled into wheel-barrows and wheeled to forms made of plank a short distance away. The specimens were cast in a horizontal position. Embedded in each for its full length and projecting from the end (forming a loop) was a $\frac{5}{8}$-in. square twisted steel bar, bent in its middle (embedded near diagonal corners), to give the specimens stiffness, so that they could be handled without breaking. There was also embedded in the upper end of each specimen for a depth of 8 ft. a steel pipe, $2\frac{1}{2}$ in. outside diameter. This was used merely to core a hole and, after the specimens were set, was removed in the following manner: Before placing, it was coated on the outside with paraffin. After the specimens were set, live steam was turned into the pipe, which then was removed easily and quickly. In the very top of each specimen was embedded a 3-in. pipe which was threaded and capped. The object in coring this hole and capping it was to see if sea water percolated into the center of specimens. (This was used afterward for fastening hooks supporting the specimens when the loops had rusted away.)

"The sand and stone were measured in accurately made frames. In measuring they rested on a perfectly smooth hardwood floor. The materials were struck smooth on top with a straight-edge. The frames were made deep in order to avoid the large errors in volume which occur when frames are very shallow and cover a large area. The sand and stone were completely fed into the mixer first; then cement was shoveled in, which took about 1 min. After $\frac{1}{2}$ min., water was added with pails, being accurately weighed. This took about 1 min. to add. In every case, in as near 2 min. as possible after the first water was added, the batch was dumped from the mixer. It took from 15 to 18 min. to place a batch entirely in the forms, into which it was thoroughly spaded and tamped. The workmanship was intended to reproduce as nearly as possible actual commercial conditions.

"The first nine specimens, which were made from a standard average composition Portland cement, were made by mixing together equal parts of Vulcanite, Alpha, and Giant. These were thoroughly mixed in the mortar box and then repacked by weight. The cement free from iron was Blanc Stainless; the commercial Portland high in alumina was Atlas; the commercial Portland cement low in alumina, was a mixture of equal parts of Lehigh and Helderberg; the iron ore cement practically free from alumina was Hermmoor Erz German Portland cement; and the Portland cement, made from blast-furnace slag and limestone, was Universal.

FIG. 1.—CONCRETE SPECIMENS SUSPENDED FROM PIER 9, BOSTON NAVY YARD.

"The stone was broken trap rock from the quarries of E. Burnett, Franklin Park ledge, Malden, Mass. A cubic foot of stone weighed 90.24 lb. Determination of voids in stone was 50 per cent. The stone passed a 1¼-in. ring, with the dust screened out.

"The sand was clean and sharp, a little finer than usual, from the bank of P. O'Riorden, at South Acton, Mass. Dry, it weighed 97.18 lb. per cu. ft. Determination of voids in the dry sand was 34.2 per cent. As used in the work, the sand contained 4.6% of moisture by weight.

"With the above determined percentage of voids, the mixture, nominally 1:2:4 but proportioned so as to have 10% excess of cement over the voids in sand, and 10% of mortar over voids in the stone, was actually proportioned 1:2½:4½.

"The specimens remained in the building for about 5 weeks. They were then loaded on flat cars and moved to Pier 9 in the Yard, and were hung from the cap log, as shown by Fig. 1. During the loading of the specimens, they were calipered in each direction close to each end and every 2 ft. along their length. Their length was also measured and the length to which the core pipe was embedded. Before hanging from the pier, they were weighed on platform scales.

"In the process of moving and setting, Specimen No. 22 was cracked clear through, at about 6 ft. from the lower end, and Specimen No. 23 had a very small crack on the face, about the middle.

"From concrete left over from filling the forms, fifteen 8-in. cubes were made of mix similar to Specimen No. 3, and fifteen others were mixed similar to Specimen No. 9. These were put in a box for convenience in keeping them together; one-third were immersed permanently; one-third were supported at half-tide; and one-third were kept in a dry place exposed to the weather."

The analyses of sand and stone made by Mr. Herbert L. Sherman for the Aberthaw Construction Company, as given in Table 1, apply to all the specimens.

TABLE 1.—Analyses of Sand and Stone Used in all Specimens.

Bank Sand.

Fineness, passing sieves, by weights.								
No. 100	No. 50	No. 30	No. 20	No. 16	No. 10	No. 8	No. 6	¼-inch.
4.8%	24.0%	52.0%	73.4%	86.4%	91.0%	93.4%	96.1%	100.0%

Voids by volume..... 34.2 per cent.
Moisture..... 4.6 " "

The sand is very fine, but is clean and of good quality.

TABLE 1.—(*Continued*).

BROKEN STONE.

Fineness, passing sieves, by weights.						
⅛-in.	3/16 in.	¼ in.	⅜-in.	½-in.	¾ in.	1-in.	1½-in.
0.25%	0.50%	1.75%	9.75%	25.25%	53.00%	80.75%	100.0%

Voids by volume..... 50.0 per cent.
The stone is a broken trap rock of very good quality.

Table 2 shows the analyses of the various cements used in preparing the specimens.

TABLE 2.—

By Mr. Herbert

Brand.....	Vulcanite, Alpha, Giant, Mixed equally. (Average alumina.)	Lehigh, Helderberg, Mixed equally. (Low alumina.)	Atlas. (High alumina.)	Blanc. (White Portland).
Percentage passing No. 100 sieve.....	94.8	93.8	94.2	94.4
Percentage passing No. 200 sieve.....	79.6	78.8	76.2	75.0
Initial set.....	1 hr. 35 min.	2 hr. 10 min.	1 hr. 45 min.	0 hr. 25 min.
Final set.....	4 hr. 45 min.	5 hr. 0 min.	4 hr. 10 min.	3 hr. 20 min.
Percentage of water.....	22	22	23	23
24 hr. neat, pounds tension.	313	312	342	277
7 days " " " "	660	686	677	549
28 days " " " "	755	741	744	726
48 hr. moist air, pounds, tension.....	.....	.....	.....	.....
Percentage of water.....	22	22	22	23
7 days, Ottawa sand, 1:3.....	264	184	272	191
28 " " " 1:3.....	360	312	304	319
Percentage of water.....	9	9	9	9
7 days, Bank sand, 1:3.....	217	133	168	159
28 " " " 1:3.....	318	271	211	218
Percentage of water.....	10	10	10	10
Loss on ignition, Percentage.	0.96	1.04	1.06	1.69
Silica (SiO_2), "	22.08	23.40	21.46	24.58
Alumina (Al_2O_3) "	7.21	5.61	8.50	8.22
Iron oxide (Fe_2O_3) "	2.37	2.37	2.44	0.29
Lime (CaO) "	62.60	62.90	61.64	62.70
Magnesia (MgO) "	2.74	2.51	2.58	1.11
Sulph. anhydride (SO_3), Percentage.....	1.56	1.53	1.75	1.32
Specimen numbers.....	1 to 9, inclusive. 20 to 24, inclusive.	14 and 15	12 and 13	10 and 11

The analysis of the water used in mixing the specimens is indicated clearly enough by the analysis of tap water at the time, as stated in the report of the Metropolitan Water and Sewerage Board. It is as follows:

Analyses of Water (Report of Metropolitan Water and Sewerage Board, 1909, p. 232.) Water taken from tap at State House, Boston, Mass., February 1st, 1909.

Parts per 100 000.

Appearance: Turbidity very slight. Sediment, slight.
Color: Platinum standard. 20.
Odor: Cold: Decided geranium, Asterionella.
Hot: " " "

Residue on evaporation: Total 3.5. Loss on ignition, 1.2.

CEMENT ANALYSES.

L. Sherman.

Universal.	Hermmoor Erz. (Iron ore.)	Vulcanite... Alpha..... Giant ..... Pulv. clay, 5%. } Mixed equally.	Vulcanite.. Alpha. ..... Giant ..... Hyd. lime, 10%. } Mixed equally. 90%	Vulcanite.. Alpha..... Giant ..... Mixed sea water. } Mixed equally.
95.6	95.8	Clay 99.0	Hyd. lime 95.2	.....
78.8	82.6	Clay 58.0	Hyd. lime 88.2	.....
35 min.	45 min.	1 hr. 40 min.	10 min.	1 hr. 15 min.
2 hr.	26 hr.	4 hr. 40 min.	4 hr.	4 hr. 30 min.
22	22	22	25	22
315	0.0 *	248	276	339
608	441	723	566	690
717	641	762	717	735
.....	223	.....	.....	.....
22	22	22	25	22
194	132	211	148	204
258	234	255	216	.....
9	10	9	9	9
166	108	169	117	223
181	174	222	182	.....
10	11	10	10	10
2.82	0.91	0.96	20.24 (18.78 water)	
20.42	24.28	22.08	1.32 and insoluble matter	
8.04	0.94	7.21 Al_2O_3	0.68	
3.04	9.08	2.57 Fe_2O_3		
62.16	62.12	62.60	45.88	
1.68	0.43	2.74	31.62	
1.73	1.75	1.56 CaO_2 (Hydrated lime.)	1.51	
18 and 19	16 and 17	24	22	21

* No strength on 24-hr. neat test; cement did not have hard set.

Ammonia:	Free, 0.0016	
	Albuminoid, Total 0.0128	Dissolved 0.0108
		Suspended 0.0020
Chlorine:	0.38	
Nitrogen: as	Nitrates, 0.0060; as Nitrites, 0.0000.	
Oxygen consumed:	0.17	
Hardness:	1.7	

The chemical analysis, by A. L. Babbitt, Navy Yard Chemist, of the salt water of Boston Harbor in which the specimens are immersed, at low tide, is as follows (Samples of sea water taken February 1st. 1916, from slip between Piers 8 and 9, for use in report of concrete test specimens):

Specific gravity	1.0220	
Total solids	32 100	parts per million.
Silica	1.6	" " "
Iron and aluminum oxides	3.6	" " "
Calcium	365.9	" " "
Magnesium	1 138.7	" " "
Sodium	9 163.0	" " "
Potassium	508.6	" " "
Sulphate radical (SO_4)	2 232.2	" " "
Chlorine	16 350.0	" " "

Hypothetical Combinations.

	Parts per million.	Grains per U. S. gallon.
Silica	1.6	0.10
Iron and aluminum oxides	3.6	0.20
Calcium carbonate	780.4	45.5
Calcium sulphate	181.2	10.6
Magnesium sulphate	2 637.7	153.8
Magnesium chloride	2 370.1	138.2
Sodium chloride	23 290.5	1 357.8
Potassium chloride	966.3	56.3

And at high tide as follows:

Specific gravity	1.0217	
Total solids	31 820.	parts per million.
Silica	1.2	" " "

Iron and aluminum oxides.....	3.6	parts	per	million.
Calcium	354.5	"	"	"
Magnesium	1 117.1	"	"	"
Sodium	9 041.0	"	"	"
Potassium	503.4	"	"	"
Sulphate radical (SO_4).....	2 286.6	"	"	"
Chlorine	15 980.0	"	"	"

Hypothetical Combinations.

	Parts per million.	Grains per U. S. gallon.
Silica	1.2	0.07
Iron and aluminum oxides.....	3.6	0.20
Calcium carbonate.....	844.2	49.2
Calcium sulphate.....	39.6	2.3
Magnesium sulphate.....	2 831.5	165.1
Magnesium chloride.....	2 132.1	124.3
Sodium chloride.....	22 980.4	1 339.8
Potassium chloride.....	956.4	55.9

Table 3 shows the scheme of the tests. For the purpose of this report, they have been arranged in eleven series, according to the cement used. Those in series II to XI, inclusive, in every case have parallel tests in Series I, which is the principal one.

Table 3-*A* gives, for each specimen:

(*a*) The important dates;

(*b*) The materials of which made;

(*c*) The proportions.

The percentage of water used, as given in the tables, is not an absolute guide to the quantity in the concrete, because, during a portion of the tests, a snow storm caused the sand and stone to become wetter initially.

The progressive condition of the specimens is shown in Table 4 where the information is collated for convenient reference.

The observations as to condition of the specimens were made at various times by Mr. E. S. Phelps, DeWitt C. Webb, M. Am. Soc. C. E., Civil Engineer, U. S. Navy, and by the writer.

TABLE 3.—

Proportions.	Series I. Mixed Cement. Average Alumina. Al_2O_3 7.21% MgO 2.74%		Series II. Blanc Cement. Al_2O_3 8.22% MgO 1.11%		Series III. Atlas Cement. Al_2O_3 8.5% MgO 2.58%		Series IV. Helderberg and Lehigh Cement. Al_2O_3 5.61% MgO 2.51%		Series V. Hermmoor Erz Iron Ore Cement. Al_2O_3 0.94% MgO 0.48%	
	No. of Specimen.	Water, in pounds per cubic foot.	No.	Water.	No.	Water.	No.	Water.	No.	Water.
1:1 :2	1	Dry 9.3								
1:1 :2	2	Wet 10.4	10	Wet 10.2	12	Wet 11.0	14	Wet 11.7	16	Wet 11.1
1:1 :2	3	Very wet 12.5								
1:2½:4½	4	Very dry 5.9								
1:2½:4½	5	Wet 7.1								
1:2½:4½	6	Very wet 12.2								
1:3 :6	7	Very dry 5.8								
1:3 :6	8	Wet 7.7								
1:3 :6	9	Very wet 11.3	11	Very wet 12.1	13	Very wet 11.3	15	Very wet 11.7	17	Very wet 10.2

Note.—The weight of water, in pounds per cubic foot, takes

A very careful examination, by three experienced observers, was made in April, 1914, at low tide, of all the specimens, and they were graded and classified on the spot in accordance with their condition at the time. The estimates of grade and class were on the basis of durability or serviceability of the concrete, as if it were in actual service. The examination was made after the specimens had been in the water 5 years, since which time there have been no changes of note. The various classes, especially the upper ones, grade into one another with definite, but without particularly strong, dividing lines. The specimens in each class are arranged from left to right in the order of durability.

The following is a copy of the report on classifying and grading the specimens:

"April 1st, 1914.

"Specimens graded largely as to condition at point of maximum deterioration.

"Specimens in order of durability.

"Elevation of tide, 1 ft. above mean low water at time of examination.

SCHEME OF TESTS.

SERIES VI. UNIVERSAL CEMENT. Al_2O_3 8.04% MgO 1.63%		SERIES VII, LIKE SERIES I, CONCRETE EXTRA WELL MIXED. Al_2O_3 7.2% MgO 2.74%		SERIES VIII, LIKE SERIES I, USING SEA WATER.		SERIES IX, LIKE SERIES I, WITH LIME.		SERIES X. LIKE SERIES I, WITH SYLVESTER SOLUTION.		SERIES XI, LIKE SERIES I, WITH 5% CLAY.		
No.	Water.	No.	Water.	No.	Water.	No.	Water.	No.	Water.	No.	Water.	
18	Wet 11.5											Series XII.
19	Very wet 13.03	20	Very wet 9.7	21	Very wet 9.7	22	Very wet 10.7	23	Very wet 9.6	24	Very wet 9.5	Series XIII.

no account of moisture in the ingredients before mixing.

Class 1: Perfect condition; No. 14.

Class 2: Nearly perfect condition; Nos. 3, 16.

Class 3: Very fine condition, but with slight deterioration; Nos. 18, 12, 10, 2.

Class 4: Very good condition, but with some deterioration; Nos. 6, 5.

Class 5: Good condition, but with deterioration apparent; Nos. 15, 20, 24, 17, 1.

Class 6: Fair condition; deterioration apparent; concrete soft in spots near low-water mark; Nos. 9, 11, 13.

Class 7: Poor condition; decided deterioration; concrete shows deterioration at low water; Nos. 4, 8.

Class 8: Very bad condition; concrete disintegrated and badly eroded above low water; Nos. 22, 19, 23.

Class 9: Exceedingly bad condition; concrete totally disintegrated and eroded above low water line, No. 7."

The following specimens showed deterioration between the 1914 and 1916 inspections, arranged in the order of greatest deterioration: Nos. 22, 19, 23, 7, 11, 4. The remaining specimens show no noticeable

TABLE 3-*A*.—Data Relating

No. of specimen.	Kind of mixture.	Dates: (All in 1909.) Of mixing.	Forms removed.	Immersed.	Kinds of cement used.	Proportions.	No. of batches.
1	Dry.	Jan. 8	Jan. 9	Mar. 1	Equal parts of Vulcanite, Alpha, and Giant.	1 : 1 : 2	3
2	Plastic.	Jan. 8	Jan. 11	Mar. 1	Equal parts of Vulcanite, Alpha, and Giant.	1 : 1 : 2	3
3	Very wet.	Jan. 9	Jan. 11	Mar. 1	Equal parts of Vulcanite, Alpha, and Giant.	1 : 1 : 2	3
4	Dry.	Jan. 9	Jan. 11	Mar. 1	Equal parts of Vulcanite, Alpha, and Giant.	1 : 2⅓ : 4⅔	3
5	Plastic.	Jan. 9	Jan. 11	Feb. 23	Equal parts of Vulcanite, Alpha, and Giant.	1 : 2⅓ : 4⅔	3
6	Very wet.	Jan. 9	Jan. 11	Feb. 23	Equal parts of Vulcanite, Alpha, and Giant.	1 : 2⅓ : 4⅔	3
7	Quite dry.	Jan. 11	Jan. 13	Feb. 27	Equal parts of Vulcanite, Alpha, and Giant.	1 : 3 : 6	4
8	Plastic.	Jan. 11	Jan. 13	Feb. 23	Equal parts of Vulcanite, Alpha, and Giant.	1 : 3 : 6	4
9	Wet.	Jan. 11	Jan. 13	Feb. 23	Equal parts of Vulcanite, Alpha, and Giant.	1 : 3 : 6	4
10	Wet.	Jan. 12	Jan. 14	Feb. 27	Blanc (white Portland).	1 : 1 : 2	3
11	Wet.	Jan. 12	Jan. 14	Feb. 27	Blanc (white Portland).	1 : 3 : 6	3¾
12	Wet.	Jan. 12	Jan. 14	Feb. 27	Atlas, high in alumina.	1 : 1 : 2	3
13	Wet.	Jan. 12	Jan. 14	Feb. 27	Atlas, high in alumina.	1 : 3 : 6	4
14	Wet.	Jan. 13	Jan. 15	Feb. 23	Equal parts of Helderberg and Lehigh.	1 : 1 : 2	3
15	Wet.	Jan. 13	Jan. 15	Feb. 27	Equal parts of Helderberg and Lehigh.	1 : 3 : 6	4
16	Wet.	Jan. 15	Jan. 18	Feb. 23	Iron ore (Hermmoor Erz) (German).	1 : 1 : 2	3

TO MIXTURE OF CONCRETE.

WEIGHTS (ALL IN POUNDS):						Volume of concrete, in cubic feet.	Percentage of water to total weight.	Percentage of water to cement, by weight.	Condition of concrete after mixing.
Of water in batches.	Total of water.	Total of cement.	Of specimen.	Per cubic foot.	Of water per cubic foot.				
88 80 80	248	1128	4057.5	152.1	9.3	26.68	6.1	22	Water glistened, but did not spatter or quake.
107 93 95	295	1128	4267.5	151.06	10.4	28.25	6.9	25.2	No water spotted surface from tamping, but a little movement could be seen 12 in. from tamp.
125 150 150	425	1128	4197.5	145.04	12.5	28.044	8.4	37.6	Soft enough to run out of overturned wheel-barrow, but would not flow like syrup. First two batches required a little tamping. Third batch showed quake when tamping.
58 55 52	165	1128	4122.5	145.82	5.9	28.267	4.0	14.6	No free water, but surface was moist. Shoveled from barrow and every shovelful tamped by two men.
66 63 66	195	1128	4127.5	150.3	7.1	27.46	4.7	17.2	Top surface quite wet. Just barely spattered, and quaked a very little at a few inches from tamp.
125 115 105	345	1128	4297.5	152.2	12.2	28.225	8.0	31.4	Nothing in report.
47 45 45 45	182	1504	4097.5	142.5	5.8	28.74	4.1	12.1	When filled glistened at a point about 4 ft. from end. Did not quake.
60 60 60 60	240	1504	4202.5	146.58	7.7	28.67	5.7	16	Moderate tamping showed quaking; spots of water on surface.
88 85 85 85	343	1504	4162.5	145.4	11.3	28.62	7.8	22.8	Forms filled and water ran off, spattered and quaked.
95 100 110	305	1128	4267.5	142.1	10.2	30.03	7.1	26.2	Quaked when tamped lightly. Quite wet when dumped. Cement acted quickly.
100 100 93 60	353	1455	4297.5	146.20	12.1	29.39	8.5	24.2	Flowed off shovel easily. Showed quaking, and spattered from light tamping. Concrete when tamped flowed water off form. This concrete set before that of No. 10 specimen made previously.
105 110 110	325	1128	4172.5	145.9	11.0	28.58	7.5	28.8	Quaked with light tamping. Spotted on surface.
95 95 95 60	345	1504	4047.5	142.8	11.3	28.34	7.9	25.1	Flowed off shovel easily. Tamping showed quaking. Spattered when tamped hard. Surface glazed. Light tamping showed quakes.
110 110 110	330	1128	4077.50	144.2	10.7	28.27	8.1	29.1	Quakes with light tamping. Spattered and showed water on surface.
95 95 95 50	335	1504	4072.50	142.7	11.7	28.52	8.2	22.1	Showed quaking, light tamping. Surface glistens.
105 105 105	315	1131	4237.5	149.6	11.1	28.32	7.4	29.2	When leveled off showed oily streaks in water on top. Very dark, more like black mud mixture. Very oily surface. Showed quake in light tamping. Very soft after 24 hours' set; finger nail made impression.

TABLE 3-A.—

No. of specimen.	Kind of mixture.	Dates: (All in 1909.) Of mixing.	Forms removed.	Immersed.	Kinds of cement used.	Proportions.	No. of batches.
17	Wet.	Jan. 15	Jan. 18	Feb. 27	Iron ore (Hermmoor Erz) (German).	1 : 3 : 6	3¾
18	Wet.	Jan. 13	Jan. 15	Feb. 27	Universal.	1 : 1 : 2	3
19	Wet.	Jan. 13	Jan. 15	Feb. 23	Universal.	1 : 3 : 6	3¾
20	Quite wet.	Jan. 14	Jan. 16	Feb. 23	Vulcanite, Alpha, and Giant, mixed.	1 : 3 : 6	3¾
21	Wet. (Sea water.)	Jan. 14	Jan. 16	Feb. 23	Vulcanite, Alpha, and Giant, mixed.	1 : 3 : 6	3¾
22	Wet. (Lime.)	Jan. 16	Jan. 18	Feb. 23	Vulcanite, Alpha, and Giant, with one-tenth hydrated lime.	1 : 3 : 6	3¾
23	Wet. (Sylvester solution.)	Jan. 16	Jan. 18	Feb. 23	Vulcanite, Alpha, and Giant, mixed.	1 : 3 : 6	3¾
24	Wet. (Clay.)	Jan. 16	Jan. 18	Feb. 23	Vulcanite, Alpha, and Giant, mixed with pulv. clay.	1 cement : 1/20 pulv. clay : 3 sand : 6 stone.	3¾

deterioration between 1914 and 1916. It is to be noted that all those that have deteriorated are in the lowest classes as to durability. Careful consideration showed that the classification made in 1914 need not be changed.

All specimens since 1914 have become coated with a fine layer of crude fuel oil between extreme high and low waters. This coating hides many of the minor defects in the specimens. The oil comes from the outlets placed on Pier 9 for fueling naval vessels.

The report of April, 1914, on the durability of the concrete, forms the basis for all the analyses of the results and for all the conclusions drawn. A tabulation and a discussion of results is offered in the following pages. Reference is made particularly to Table 3, showing the scheme of the tests and the division of the tests into thirteen series, each devised to show the effect of some particular condition, material,

(*Continued*).

Weights (All in Pounds): Of water in batches.	Total of water.	Total of cement.	Of specimen.	Per cubic foot.	Of water per cubic foot.	Volume of concrete in cubic feet.	Percentage of water to total weight.	Percentage of water to cement, by weight.	Condition of concrete after mixing.
80									
80	293	1417	4172.5	145.9	10.2	28.59	7.0	20.8	Water showed on oily surface. Quaked on tamping. After 3 days showed a crust on surface.
53									
110									
110	330	1128	4162.5	145.6	11.5	28.58	7.9	29.2	Quaked with light tamping. Very dark in color.
110									
100									
100	370	1410	4072.5	143.3	13.3	28.43	8.6	25.6	Quaked from light tamping. Showed water on surface, not enough to spatter from tamping.
70									
75									
75	275	1410	4112.5	145.5	9.7	28.26	6.7	19.5	Showed quaking and spattered when spading sides. Water flowed over surface and down sides of forms.
50									
75									
75	275	1410	4147.5	147.0	9.7	28.21	6.6	19.5	Showed quake; water flowed over surface.
50									
90									
95	325	1410	4172.5	146.7	10.7	28.43	7.3	22.7	Showed quake on light tamping; also spattered. Specimen cracked in center.
90									
50									
80									
80	298	1410	3997.5	137.9	9.6	28.97	7.0	21.2	Quaked and spattered when tamping. Very fine cracks near middle.
85									
53									
75									
75	275	1410	4097.5	151.6	9.5	26.97	6.2	19.5	Showed quaking, and spattered from light tamping.
75									
50									

or process. The scheme of tests is as planned by those who originally laid them out; the table is devised for the purpose of this paper.

The following important results are shown by an examination of Table 5:

(*a*) The effect of proportions of materials is the most marked phenomenon developed in the tests.

(*b*) In every instance the specimen richer in cement is more durable than the corresponding leaner specimen, which is shown in detail by referring to Table 3 in connection with Table 5, as follows:

No. 1 is better than No. 4, which in turn is better than No. 7.
" 2 " " " " 5 " " " " " " 8.
" 3 " " " " 6 " " " " " " 9.
" 10 " " " " 11

TABLE 4.—(*Continued.*)

February, 1909.	March, 1910.	March, 1911.	June, 1912.	November, 1912.	December, 1913.	April, 1914.	January, 1916.
14	No action.	No action.	No action.	No action.	No action.	No action.	No action.
15	Fair; slight action on front.	No action, except slightly rough on face.	Back slightly flaked.	Slight deterioration, face and side.	Faces slightly marked.	Concrete slightly soft in spots. Face and sides pitted; slight erosion.	Concrete soft in spots. Face and sides pitted; slight erosion.
16	No action.	No action.	No action.	No action.	No action.	Very slight pitting at extreme low water.	Very slight pitting at extreme low water.
17	No action.	No action.	All sides flaked.	All sides flaked.	All sides very slightly flaked.	All sides pitted. Front badly pitted at M. L. W.	All sides pitted. Front badly pitted at M. L. W.
18	No action.	No action.	No action.	No action.	No action.	Sides perfect. Front and back slight pitting.	Sides perfect. Front and back pitted.
19	Very good. Face slightly rough.	No action, except slightly rough on face; cracked.	Face eroded. Concrete soft.	Back disintegrated; concrete very soft.	Badly flaked; concrete crumbling back 4-in. hole.	Face badly pitted; concrete soft; back eroded 4 in. deep.	Concrete soft and spongy; face pitted; sides badly eroded.
20	Very good. Face slightly rough.	No action, except slightly rough on face; cracked.	No action, except back flaked.	Slight deterioration at M. L. W.	Flaking all sides; corners worn.	Concrete fairly hard. Face eroded. Sides and back somewhat pitted.	Concrete fairly hard. Face eroded. Sides and back somewhat pitted.
21	Very good. Face slightly rough.	No action, except slightly rough on face; cracked.	Sunken.	Lost.	Lost.	Lost.	Lost.
22	Good; crack on face.	Rough on face.	Face and back eroded.	Face and sides eroded; edges broken.	Badly marked; 4-in. hole on face; concrete soft and dead.	Face eroded 6 in. over space 3 ft. Sides and back pitted. Concrete soft.	Face eroded 7 in., length 4 ft. Sides and back eroded from 1 to 3 ft. Concrete soft.
23	Cracked. Corners broken.	Cracked. Corners broken.	Face and back edges eroded.	Edges badly broken away.	Face badly flaked; edges bad.	Concrete eroded from 4 to 8 in. at M. L. W. Soft.	Concrete eroded from 5 to 8 in. deep about M. L. W. Concrete soft below M. L. W.
24	Good. Face slightly soft.	No action.	No action, except edges eroded.	Back edges worn; mortar on face deteriorated.	Face slightly marked. Edges worn.	Faces and sides pitted. Edges slightly soft on lower half Spec. Concrete fairly hard.	Faces and sides pitted. Edges slightly soft on lower half. Concrete hard.

TABLE 4.—Progressive Condition of Specimens.

February, 1909.	March, 1910.	March, 1911.	June, 1912.	November, 1912.	December, 1913.	April, 1914.	January, 1916.
1	Face rough.	Face rough, 1 in. deep.	Face rough, 1 in. deep.	Face eroded, 1 in. deep.	Face badly eroded.	Face badly eroded; porous spots.	Face badly eroded; some porous spots.
2	No action.	No action.	No action.	Slightly rough face; slightly porous.	Faces slightly pock-marked.	Edges slightly soft; faces practically perfect.	Edges slightly soft; faces practically perfect.
3	No action.	No action.	Sunken.	Slightly rough face; scale on back.	Face and back slightly pock-marked.	All faces very good; surface slightly pitted; slight injury to edges.	Faces very good; surface slightly pitted. Slight injury to edges.
4	Face pitted 1 in., sides less.	Face pitted 1 in., sides less.	Sunken.	Very rough face; concrete porous.	Face badly eaten; back flaked; sides rough.	Face badly eroded; little pitting and erosion.	Face badly eroded; some pitting and erosion.
5	No action.	No action.	No action.	Face slightly rough; porous spots on back.	Faces (all) pock-marked.	Some erosion on face; pitting on one side.	Some erosion on face; pitting on one side.
6	Very good. Surface soft.	No action.	No action.	Edges slightly broken.	Fine condition. Corners worn; face marked.	Edges show action. Face pitted; soft spots on back.	Edges show action. Face pitted; soft spots on back.
7	Face pitted; sides less.	Face eroded; sides good.	Considerable erosion, front and back.	Badly eroded, back and face; corners soft and crumbling.	Very poor; concrete soft and crumbling.	Very poor condition; 2½ in. in length completely gone.	Very poor condition; 5 ft. in length completely gone.
8	No action.	No action except back eroded.	Back eroded 4 in; concrete soft.	Back disintegrated. Sides and face rough. Concrete soft.	Front and sides marked. Back 5-in. hole.	Face and sides slightly pitted. Back eroded 4 in. deep.	Face and sides slightly pitted. Back eroded 5 in. deep; rods exposed.
9	No action.	No action.	No action, except back chafed.	Edges worn. Slightly rough face.	Face marked. Edges worn.	Face badly pitted; sides and back slightly pitted.	Face badly pitted; side and back slightly pitted.
10	No action.	No action	No action.	Slightly rough face.	Face marked.	Some erosion on face; side and back slight pitting.	Some erosion on face; sides and back slight pitting.
11	No action.	No action.	No action.	Slightly rough face.	Face and sides marked.	Concrete hard, but chalky on surface. Edges soft. Face and sides very good.	Concrete chalky on surface. Edges soft. Face and sides good.
12	Very fair. Small areas on face have action noticeable.	No action, except slight roughness on face.	No action, except back slightly peeled.	Slightly rough face. Porous spots on back.	Back and face very slightly marked.	Very slight pitting on back; other faces fine.	Slight pitting on back; other faces fine.
13	Very slight action on face.	No action, except slight roughness on face.	No action, except slight erosion on back.	Slight disintegration of face.	Face marked; edge worn 2 in. deep.	Face pitted entire length; edges soft. Back slight pitting.	Face pitted entire length; edges soft Back slight pitting.

No. 12 is better than No. 13
" 14 " " " " 15
" 16 " " " " 17
" 18 " " " " 19

(*c*) Only specimens of 1:1:2 mixture appear in the first three classes.

(*d*) The specimens of $1:2\frac{1}{2}:4\frac{1}{2}$ (the next richer) mixture appear in Class 4.

(*e*) The one specimen of 1:1:2 mixture, *viz.*, No. 1, and those specimens of $1:2\frac{1}{2}:4\frac{1}{2}$ mixture which fall below the others of their class, evidently do so from being mixed dry.

(*f*) The poorest specimens were of 1:3:6 mixture.

TABLE 5.—CLASSIFICATION OF CONCRETE SPECIMENS, AFTER 5 YEARS OF EXPOSURE IN SEA WATER, IN THE ORDER OF DURABILITY, SHOWING THE INFLUENCE OF THE PROPORTIONS OF MATERIAL.

Analysis of Series I to VII, inclusive, Table 3.

1:1:2 mixtures marked ○
$1:2\frac{1}{2}:4\frac{1}{2}$ mixtures marked □
1:3:6 mixtures not marked.

Durability.	Specimen No.				
Class I...	⑭				
Class II...	③	⑯			
Class III...	⑱	⑫	⑩	②	
Class IV...	□6	□5			
Class V...	15	20	24	17	①
Class VI...	9	11	13		
Class VII...	□4	8			
Class VIII...	22	19	23		
Class IX...	7				

The important effects of the quantity of water used in mixing are as follows:

(*a*) The weight of water, in pounds per cubic foot of concrete, takes no account of moisture in the ingredients before mixing.

(*b*) Specimens 1, 2, and 3 were made in order to test the effect of varying proportions of water. The results show durability—least with the dry and greatest with the very wet.

(*c*) Specimens 4, 5, and 6, also made to test the effect of varying proportions of water, show least durability with the dry and greatest with the very wet, this being slightly better than the wet.

(*d*) Specimens 7, 8, and 9 were also made to test the effect of varying proportions of water, and again show least durability with the dry and greater with the very wet specimen.

(*e*) In general, the results show that dry mixtures are of the least value, and that wet or very wet mixtures decidedly improve the durability.

TABLE 6.—Classification of Concrete Specimens, After 5 Years of Exposure in Sea Water, in the Order of Durability, Showing the Influence of the Quantity of Water Used in Mixing.

Analysis of Series I to XIII, inclusive, Table 3.

Specimens of a dry mixture marked D.

Specimens of a wet mixture marked W.

Specimens of a plastic mixture marked P., equals wet W.

Specimens of a very dry mixture marked V. D.

Specimens of a very wet mixture marked V. W.

Pounds of water per cubic foot of concrete shown below the number.

Durability.	Specimen No.				
Class I...	W. $\frac{14}{11.7}$				
Class II...	V. W. $\frac{3}{12.5}$	W. $\frac{16}{11.1}$			
Class III...	W. $\frac{18}{11.5}$	W. $\frac{12}{11.0}$	W. $\frac{10}{10.2}$	P. $\frac{2}{10.4}$	
Class IV...	V. W. $\frac{6}{12.2}$	P. $\frac{5}{7.1}$			
Class V...	W. $\frac{15}{11.7}$	V. W. $\frac{20}{9.7}$	$\frac{24}{9.5}$	W. $\frac{17}{10.2}$	D. $\frac{1}{9.8}$
Class VI...	W. $\frac{9}{11.3}$	W. $\frac{11}{12.1}$	W. $\frac{13}{11.3}$		
Class VII...	D. $\frac{4}{5.9}$	P. $\frac{8}{7.7}$			
Class VIII...	W. $\frac{22}{10.7}$	W. $\frac{19}{13.3}$	$\frac{23}{9.6}$		
Class IX...	V. D. $\frac{7}{5.8}$				

TABLE 7.—Classification of Concrete Specimens, After 5 Years of Exposure in Sea Water, in the Order of Durability, Showing the Influence of the Quantity of Water Used in Mixing.

Analysis of Series I (Table 3).

Durability.	Very wet.	Wet.	Dry.
Class I			
Class II	(3)		
Class III		(2)	
Class IV	[6]	[5]	
Class V			(1)
Class VI	9		
Class VII		8	[4]
Class VIII			
Class IX			7

Table 7 shows more clearly than Table 6 the effect of varying quantities of water used in mixing, because only those specimens are included which were devised to test this particular feature. The results were as follows:

(*a*) The bad effect of a "dry" mixture is plainly shown. In each of the three series the dry specimen is the least durable and the very wet the most durable.

(*b*) The 1:1:2 mixture is the most durable of each of the three classes of specimens, the dry, the wet, and the very wet.

(*c*) The 1:2½:4½ mixture is more durable than the 1:3:6 in each of the three classes of specimens, the dry, the wet, and the very wet.

(*d*) Dryness of mixture has caused one of the specimens (No. 1), rich in cement, to be less durable than two (Nos. 5 and 6), with a lesser proportion of cement. Also, a dry specimen having a medium quantity of cement (No. 4) proved less durable than a wet specimen with less cement.

The following are the important results developed, as shown by Table 8:

(*a*) It happens that the two most durable specimens are those lowest in alumina content, and, above a percentage of 7 in alumina content, no relation to durability is apparent.

(*b*) The relation of magnesia content to durability is not established, although it is noted that the first three specimens have a lower total magnesia content than the last three; but this condition is not borne out in Series XIII, Table 9.

(*c*) It is a curious condition that in the two best specimens the relation between the alumina content and the magnesia content is 2.2, the lowest of any.

TABLE 8.—Classification of Concrete Specimens, After 5 Years of Exposure in Sea Water, in the Order of Durability, Showing the Effects of the Alumina and Magnesia Content.

Analysis of Series XII (1:1:2 mixture), Table 3.

Order of Durability.		Al_2O_3.	MgO.	Total Al_2O_3 and MgO.	Factor Al_2O_3 divided by MgO.
Class.	No.				
I.....	14	5.61%	2.51%	8.12%	2.2
II.....	16	0.94	0.43	1.37	2.2
III.....	18	8.04	1.63	9.64	4.9
	12	8.50	2.58	11.08	3.3
	10	8.22	1.11	9.33	7.4
	2	7.21	2.74	9.95	2.6

TABLE 9.—Classification of Concrete Specimens, After 5 Years of Exposure in Sea Water, in the Order of Durability, Showing the Effect of the Alumina and Magnesia Content.

Analysis of Series XIII (1:2½:4½ mixture), Table 3.

Order of Durability.		Al_2O_3.	MgO.	Total Al_2O_3 plus MgO.	Factor Al_2O_3 divided by MgO.
Class.	No.				
V.....	15	5.61%	2.51%	8.12%	2.2
	17	0.94	0.43	1.37	2.2
VI.....	9	7.21	2.74	9.95	2.6
	11	8.22	1.11	9.33	7.4
	13	8.50	2.58	11.08	3.3
VIII.....	19	8.04	1.63	9.67	4.9

The following are the important results developed as shown by Table 9:

(*a*) It happens that the two most durable specimens, as in Table 8, are those lowest in alumina content. Again, above a per-

centage of 7 in alumina content, no definite relation to durability is apparent.

(*b*) There is no apparent relationship of magnesia content, within the limits obtained, to durability.

(*c*) It is a curious condition that in the two best specimens the relation between the alumina content and the magnesia content is 2.2, the lowest of any.

TABLE 10.—Classification of Concrete Specimens, After 5 Years of Exposure in Sea Water, in the Order of Durability, Showing the Durability of the Specimens as Related to Cement Mixtures.

Analysis of Series XII and XIII, Table 3.

Class.	Specimen No.	Cement.	Rank.
I	14	Lehigh and Helderberg mixed.	1
II	16	Hermmoor Erz	2
III	18	Universal	3
	12	Atlas	4
	10	Blanc	5
	2	Vulcanite, Alpha, Giant mixed	6
IV			
V	15	Lehigh and Helderberg.	1
	17	Hermmoor Erz	2
VI	9	Vulcanite, Atlas, Giant mixed	3
	11	Blanc	4
	13	Atlas	5
VII			
VIII	19	Universal	6

A study of the preceding tables discloses the results of tests of Series I to VI, inclusive, and of Series XII and XIII of Table 3. There remain the special series, numbered VII to XI, inclusive.

Specimen No. 20, Series VII, is like No. 9, Series I, except that greater care was used in mixing the ingredients. The notes say: "Better mixture than commercial 1:3:6." The results indicate that this caused it to appear in the class above its mate, it being the fourth specimen above it in durability. This specimen was better than any other in the whole of Series XIII except No. 15, which appears to have been favored by a low alumina cement. The extra care used in mixing showed a decidedly beneficial result.

Specimen No. 21, Series VIII, was made like No. 9, but sea water was used in mixing. Unfortunately, this specimen was lost overboard,

and could not be recovered. At the end of 2 years it was in good condition and equal to No. 9, except for some slight roughness on the face. Evidently the use of salt water did no material injury, and was not as deleterious as the use of insufficient fresh water.

Specimen No. 22, Series IX, was mixed with 10% of hydrated lime substituted for an equal quantity of cement; otherwise, it was the same as No. 9. The durability tests show it to be two classes below No. 9, or the fifth below it serially in grade. Evidently, the lime allowed an earlier and more rapid erosion. This specimen is also poorer than eight of the other ten similar specimens in series, showing conclusively, so far as one specimen can, the deleterious effects of the hydrated lime when used in concrete immersed in sea water.

Specimen No. 23, Series X, contained the same ingredients as No. 9 with the addition of Sylvester wash. The water used in mixing contained soap solution made by dissolving light-colored soap in the proportion of 1¼ lb. to 15 gal. (125 lb.) of water. To each bag of cement was added 3 lb. of powdered alum. The total quantity of water in the batch was 288 lb. and of cement 5 bags. The addition of the Sylvester, according to the results of the tests, was decidedly harmful, as this specimen gave the poorest result of eleven similar ones. The concrete at the end of 5 years was in very bad condition.

Specimen No. 24, Series XI, is the same as No. 9, except that to each bag of cement there was added 4¾ lb. of clay. The clay was taken from a near-by excavation, dried, powdered, and mixed with each bag of cement. This specimen came through in good condition—better in fact than its mate, No. 9, and better than all the other specimens of a similar mixture, save only No. 15 (low alumina cement) and No. 20 (extra well-mixed concrete). There is evidence, therefore, that the addition of 5% of clay to the cement had a beneficial effect.

On September 11th, 1916, R. J. Wig, Assoc. M. Am. Soc. C. E., Engineer-Physicist of the Bureau of Standards, Washington, D. C., accompanied by L. R. Ferguson, Assoc. M. Am. Soc. C. E., representing the Association of Cement Manufacturers, visited the Boston Navy Yard in connection with studies of action of concrete in sea water. They were requested by the writer and kindly consented to examine the concrete specimens and to rate them in accordance with their condition at the time of the examination. It may be noted that the writer's rating of the specimens was made more

than 2 years before. Only very minor differences in the two sets of ratings occurred. In the specimens of greatest deterioration these differences can be explained by the changes in rate of deterioration during the 2-year period. The minor differences in the better specimens is explained by possible change in rate of deterioration or differences in judgment in rating two specimens nearly alike. The result as a whole is a very close confirmation of the original rating. The observers desired to note that their examinations of September 11th, 1916, were made on only the front faces and two sides of the specimens, as it was impossible to see the back faces without a small boat which was not available at the time.

The ratings, as made by Messrs. Wig and Ferguson, are as follows:

Group 1: Good condition, showing practically no deterioration; Nos. 14, 3, 16, 12, 10, 2.

Group 2: Satisfactory condition, showing little disintegration, but specimens are structurally sound; Nos. 6, 15, 18, 17.

Group 3: Poor condition, showing marked disintegration and, if part of a structure, in need of repair at once or at an early date; Nos. 1, 9, 5, 20; Nos. 4, 24, 11, 13.

Group 4: Bad condition and structurally unsound; Nos. 8, 23.

Group 5: Total failure; Nos. 22, 19, 7.

The bottoms of the specimens are at such an elevation that they are always submerged in sea water. The only chance for the examination of these parts of the specimen is when they are removed from their normal positions. On December 9th and 10th, 1913, the specimens were rehung from Pier 9. At that time the bottom of each specimen was examined, and no deterioration in the concrete was found to have taken place in any of the specimens.

The tops of all specimens are at such an elevation that they are very seldom submerged, and then only for a very short time. The concrete on the tops of all specimens appears to be in as good condition as when the specimens were originally constructed, and to show no deterioration, even in specimens that have eroded badly lower down. There are signs of some mechanical wear on the top edges of Specimens Nos. 7 and 22, but there is no appearance of deterioration of the concrete.

The 8-in. cubes formed at the time the larger specimens were made are now as follows: There are six 8-in. cubes on the quay wall adjacent to Pier 9 which are exposed to the weather in a roughly constructed box. No deterioration whatever is apparent in any of these specimens. There are five 8-in. cubes in a single box placed in the dock south of Pier 9 at the elevation of mean tide. These specimens are submerged approximately 12 hours each day. All the specimens except one show a disintegration of from 1 to 3 in. on the top, and the mortar on all but one of them is soft and crumbling to the touch, having the appearance of being dead. There is one specimen, however, that shows but little deterioration; the mortar remains hard, holding the aggregate firmly together. The top and one side only of each cube is exposed, the other sides being protected by the sides of the box and the adjacent cube. The 8-in. cubes that were originally suspended so as to be entirely submerged at all times have become lost owing to the breaking of the suspension cable, and no report can be made on them.

Conclusions.

The facts in connection with these tests have now been presented very fully, at the risk even of over-burdening the reader with details, but with the definite purpose in mind of allowing him to draw his own conclusions in the light of his own and of others' experiences. The series of experiments has been very complete in certain directions and was conducted from the start with unusual care, and some very valuable conclusions may be established. Other conclusions of less importance may also be drawn and be established with confirmatory experiences elsewhere. Great care must be exercised in applying these conclusions in predicting results elsewhere under dissimilar conditions, and the limitations under which these tests were conducted must be borne in mind.

It is desired to acknowledge valuable assistance rendered by employees of the Public Works Department, Navy Yard, Boston, in connection with observations and the preparation of this paper, especially Mr. E. S. Phelps.

APPENDIX A.

FORM OF AGREEMENT, INCLUDING SPECIFICATIONS, FOR TEST OF CONCRETE SPECIMENS, TO BE CARRIED OUT AT THE NAVY YARD, BOSTON, MASS.

This Agreement, mutually entered into between the United States Navy Department, party of the first part, and the Aberthaw Construction Company, a corporation of Maine, having a usual place of business in Boston, party of the second part, *Witnesseth,* that the said party of the second part agrees to build at the Boston Navy Yard a series of specimens to test the durability of concrete in sea water, said specimens to be built in accordance with the specifications hereto attached.

It is agreed that the party of the second part is not to put the Government to any expense in connection with these tests, and the party of the first part does not guarantee that the piers may not be disturbed as exigencies of the Navy Yard interests may require. As these experiments are a matter of considerable interest to the Government, its representatives will endeavor to continue the tests as long as it may be possible so to do.

The party of the second part agrees that all work shall be done in the presence of a representative of the Government, and that due notice of the intention to undertake work shall be given to the Commandant and to the Civil Engineer of the Yard in time for them to make arrangements for a representative to be present.

In witness whereof, the said parties have interchangeably signed and sealed this agreement on the 28th day of December, 1908.

FRED THOMPSON,
Civil Engineer, U. S. N.
ABERTHAW CONSTRUCTION CO.,
by L. C. Wason, Pres.

SPECIFICATIONS FOR TEST SPECIMENS OF CONCRETE TO BE IMMERSED IN SEA WATER BY THE UNITED STATES NAVY DEPARTMENT, DECEMBER, 1908.

Object.—The object of these tests is to determine what the action of sea water is upon concrete, both as regards climatic conditions and chemical action. The tests are to be made as nearly as possible in conformity with usual commercial work, in order to be comparable in all respects with actual work.

How Done.—The specimens are to be built in accordance with the requirements of the Navy Department, and under their direct supervision, and in accordance with the following detailed specifications.

They are to be built in molds in the sea or in such other position as may be directed, and exposed for easy inspection and for photograph-

ing. They shall be marked so as to be easily identified, and a careful record of the materials and methods used shall be kept for each specimen.

Specimens.—The piers shall be made 16 in. square and 16 ft. long, in such a position that the lower 2 ft. shall be permanently immersed in sea water; thus it is improbable that the top will ever be immersed.

There are to be twenty-four (24) piers, as follows: In the first series of nine specimens, a standard average composition Portland cement shall be used throughout, which shall pass the standard specifications of the American Society for Testing Materials, as required by the Navy Department.

No. 1 shall be made of 1 part of cement, 1 part of sand, and 2 parts of stone, and mixed quite dry.

No. 2 shall be made of the same mixture as No. 1, and mixed with sufficient water to make the concrete plastic.

No. 3 shall be the same as No. 1, but mixed very wet.

No. 4 shall approximate 1 part of cement, 2 parts of sand, 4 parts of stone, but shall be proportioned so that, after a mechanical analysis of the materials, the excess of cement over the voids of the sand shall be 10%, and the excess of mortar over the voids of the stone shall also be 10 per cent. It shall be mixed quite dry.

No. 5, same proportions as No. 4, but mixed plastic.

No. 6, same proportions as No. 4, but mixed very wet.

No. 7, 1 part of cement, 3 parts of sand, and 6 parts of stone. Mixed quite dry.

No. 8, same as No. 7, but mixed plastic.

No. 9, same as No. 7, but mixed wet.

No. 10 shall consist of a Portland cement which is free from iron. One specimen shall be mixed, of the proportions 1:1:2, quite wet.

No. 11 same cement as No. 10, 1:3:6, wet.

No. 12 commercial Portland cement high in alumina, mixed 1:1:2, wet.

No. 13 same cement as No. 12, 1:3:6, wet.

No. 14 of a commercial Portland cement low in alumina, 1:1:2, wet.

No. 15 same cement as No. 14, 1:3:6, wet.

No. 16 of an iron ore cement practically free from alumina, 1:1:2, wet.

No. 17 same cement as No. 16, 1:3:6, wet.

No. 18 of Universal cement, 1:1:2, wet.

No. 19 same cement as No. 18, 1:3:6, wet.

No. 20 shall consist of the same materials and proportions as given for No. 7, but shall be most thoroughly well mixed (much better than commercial mixing), at the same time being quite wet.

No. 21 shall be the same as No. 7, but mixed with sea water, quite wet.

No. 22 shall be mixed of $\frac{9}{10}$ by weight of one part of standard Portland cement as No. 7, $\frac{1}{10}$ part by weight of hydrated lime, 3 parts of sand, 6 parts of stone, mixed wet.

No. 23 shall be of the materials given for No. 7 but in addition shall contain Sylvester mortar hereinafter described, mixed wet.

No. 24 shall be of the materials in No. 7 and in addition shall contain 5% by weight of the cement of finely pulverized clay, mixed wet.

In addition, there shall be fifteen cubes, 8 in. on each side, of 1 part of standard Portland cement, 1 part of sand, and 2 parts stone; and fifteen others of the same cement, mixed 1:3:6. One-third of these, after being thoroughly set, shall be permanently immersed in water; another third shall be supported at about half tide; and the rest shall be kept permanently dry but exposed to the weather.

There shall also be made briquettes, of the same cements and proportions of mortar as above described for concrete, of such number that they can be tested at intervals covering a period of years, some of which shall be kept in the laboratory, some exposed to the same conditions that the cubes will have; and enough briquettes shall be made of standard sand to compare the relative strength between standard and the materials actually used.

Testing of Materials.—All cement shall be thoroughly tested for all physical properties, and shall be subjected to chemical analysis. The sand shall be subjected to a thorough mechanical and physical analysis to ascertain the relative sizes of its grains and the quantity of foreign matter which it contains, if any. The same tests shall be applied to the stone. The various ingredients, such as sea water, hydrated lime, Sylvester mortar, and clay, shall be tested, in order that the exact nature of the material may be known.

Sand.—The sand shall be of a good quality, commercial bank sand, clean, coarse and sharp, as free from all impurities and foreign matter as can be obtained commercially. The stone shall be of trap rock which shall have passed a 2-in. ring and have been retained on a $\frac{1}{4}$-in. ring. It shall be as free from dust and dirt as commercially practicable.

Mixtures.—All materials shall be proportioned by volume, as given above for each specimen. A cubic foot of cement shall contain 100 lb. The sand and stone shall be measured on the basis that a barrel of cement contains 380 lb. net and measures 3.8 cu. ft. Samples of the sand and stone shall be weighed so that the proportion by weight (as well as by volume) will be known.

The quantity of water to be used shall be accurately measured for each batch, and shall be proportioned by experiment, so that the dry specimens shall just fail to show moisture when tamped in place; the

plastic specimens shall just barely begin to quake when tamped; and the wet specimens shall be proportioned so that the mortar will flow easily off the blade of a shovel and shall be wet enough to flow readily into place without being spaded or tamped.

The Sylvester mortar shall be mixed as follows, as given in Gillette's "Hand Book of Cost Data", pages 389 and following: A light-colored soft soap shall be dissolved in water, 1¼ lb. to 15 gal. of water, and 3 lb. of powdered alum shall be mixed with each bag of cement. After all the materials of the concrete are thoroughly mixed dry, water containing the soap in solution shall be used to mix the materials wet to the proper consistency.

Mixing.—All materials shall be thoroughly mixed in a batch concrete mixer of approved type. They shall be fed into the machine as quickly as possible, and shall remain for 2 min. after the last ingredient is added. For the specimen which is to be extra thoroughly well mixed, the material shall remain in the mixer for 12 min.

If the weather is cold, all frost shall be removed from the sand and stone by heating before the materials are placed in the mixer.

Placing.—The concrete, after mixing, shall be placed in the forms as quickly and continuously as possible, and, after proper tamping, shall not be again disturbed. In no case shall concrete which has been allowed to stand for more than ½ hour be used, and in no case shall ½ hour elapse between placing two batches of concrete in the same mould.

Forms.—The forms shall consist of planed and matched spruce lumber, so that they may be as tight as possible, and shall be braced so thoroughly as not to spring under the pressure of green concrete. The forms shall be left on for such a length of time that there shall be no damage to the concrete when they are removed. The Inspector shall be the judge as to when it is proper to remove the forms.

Reinforcement.—The columns shall be reinforced with an embedded steel bar, about § in. square, which shall be bent into a U-shape and run throughout the length of the big specimens near two diagonally opposite corners, and the loop shall project a sufficient distance above the top of column to permit of its being easily hooked in case it is desired to remove it.

A hole, 3 in. in diameter, shall be cored in the upper 8 ft. of each specimen, to permit of examination from time to time to see if water has penetrated through the walls of the concrete to the hole.

Protection.—After the specimens are cast, they shall be protected from freezing, as far as possible, until they have set for a period of at least 5 days, and, after the forms have been removed, they shall be thoroughly and substantially braced, so that there shall be no danger of their tipping over.

DISCUSSION

Mr. Thomson.

T. KENNARD THOMSON,* M. AM. SOC. C. E.—This is an interesting paper on an interesting subject, and should bring out a full discussion from experience in actual work.

The author calls attention to the fact that a mixture of one part of cement to two parts of sand gave a better test than the mixtures which were not so rich in cement. This is undoubtedly due to the fact that a 1:2 mixture comes nearer to having the voids of the sand filled with cement than a poorer mixture, and, of course, a concrete with all the voids filled has a better chance of resisting the effects of frost and of any injurious chemical which may be in the water or air; and probably explains why the greatest damage to concrete is generally between high and low water.

Some years ago the speaker built a breakwater, off New Rochelle, which is subjected to storms with the full sweep of Long Island Sound behind them. A rock fill was used, up to the low-water mark, the depth of water at low water being 25 ft. at the outer end.

From low water upward the stones were carefully placed to a height of 14 ft. (except for the outer 50 ft.), on top of which a 12-in. concrete coping was placed, as individual stones would have been displaced by the storms, although the top of the breakwater was 9 ft. above ordinary high tide.

At the outer end, for 50 ft., the built-up stonework was only 8 ft. above low water, on top of which was placed a concrete block, 7 ft. high, 10 ft. wide on the base, 5 ft. on the top, and 50 ft. long; a small quantity of reinforcement was used.

This breakwater has stood successfully the severe storms of the Sound for some 11 years; but this year it has been noticed that a small quantity of the concrete has seriously deteriorated for a foot or so at the bottom and for about a foot in depth.

It was a little surprising, however, that this destruction occurred on the inside or sheltered portion of the breakwater, instead of outside, where it was exposed to the storms. This may have been due to the fact that on the exposed side of the breakwater any injurious chemicals from sewage, etc., would be washed away instead of remaining in place, as they would on the sheltered side.

It has been found impossible to get a concrete to withstand the action of impure, hot water, in some power-plants. In one case the efforts have been abandoned, and a wooden lining has been substituted for the concrete.

It would seem that the three principal causes for disintegration of ordinarily good concrete in sea water are: first, too coarse a mixture,

* New York City.

in which the voids of the sand or stone are not filled—thus allowing chemicals to accumulate; second, alternate freezing and thawing, which do far more damage when the voids are not filled; and third, the action of injurious chemicals assisted by heat. Mr. Thomson.

The remarks in the discussion about sand recall an attempt of a railroad contractor, many years ago, to make the railroad haul free of charge for 110 miles a fine beach sand, because the local sand appeared to contain too much loam. The speaker made three sets of tests: first, with regular testing quartz; second, with the loamy sand; and third, with the fine beach sand. The tensile strength developed by the briquettes of the quartz and loamy sand were almost the same, and those made with the clean but fine sand gave only half the strength of the others. As strength in the concrete was what was wanted, the work was finished with the loamy sand.

J. J. Yates,* Assoc. M. Am. Soc. C. E.—The question of the action of sea water on concrete is of great importance, as it affects the permanency of such a large number of important structures which have been, or may be, built in sea water. This paper adds much valuable information on a subject that has received far too little study. Mr. Yates.

The speaker, however, thinks that one important phase which has not been considered is injury to concrete structures by floating ice or floating débris, resulting in the breaking up of the surfaces and the exposure of the interior concrete to the action of sea water.

From observations of existing structures, extending over a period of about 15 years, the speaker is led to the opinion that, in setting, a surface is formed on concrete which will resist successfully the action of sea water on the cement; and that, as long as this surface is maintained in good condition, no destructive action due to sea water may be expected. In other words, he is of the opinion that concrete surfaces are first injured or broken up by drifting material, and that, after they are thus broken up, the concrete is not able to resist the action of the sea water. Observations indicate that this action, after it once starts, continues, and is accelerated by the wearing away of the disintegrated surfaces by drifting ice and other material, or by the alternate freezing and thawing of the concrete surfaces between tides. By one of these means, the disintegrated concrete is removed, continually exposing a fresh surface to the action of the sea water.

As examples of the action of sea water on concrete, the following are given:

The concrete masonry of the old center piers of the Hackensack and Passaic River Bridges of The Central Railroad Company of New Jersey were constructed about 1870 of Rosendale cement. The

* New York City.

Mr. Yates.

concrete of these piers was deposited in a circular, wrought-iron, floating caisson, which was sunk in position to a depth of about 12 ft. on a previously prepared pile foundation. About 1888 or 1889 the iron superstructure was renewed, making use of the old piers. When inspected about 1902 it was found that the iron caisson had completely disappeared, to a depth of several feet below low water, and that the concrete had been eaten away between high and low water to a depth, in some cases, of about 2 ft. These piers were repaired by patching with concrete behind a steel plate band extending around each pier.

The piers were not exposed to drifting material, as they were protected or practically enclosed by pile and timber fenders of a type generally used at draw-bridges. Considerable trouble was experienced, however, from ice forming around the piers. In fact, during extreme high tide conditions, the operation of the lower of the draw-bridges was seriously interfered with by ice clogging the bearing wheels on the center pier.

In 1912 both the substructure and the superstructure were removed, and temporary structures were built at one side for the purpose of building new bridges.

During 1903 to 1905 The Central Railroad Company of New Jersey replaced its draw-bridge over Newark Bay, about ¾ mile from its mouth or junction with Staten Island Sound, with two bascule spans. The seven concrete piers required were deposited in air within wooden floating caissons which were sunk to a depth of about 30 ft. on prepared pile foundations. Sea water was not allowed to come in contact with the concrete until it had had time to become thoroughly set.

The concrete was composed of 1 part of cement, 3 parts of sand, and 5 parts of 1½-in. broken trap rock. The following is a representative test analysis of the cement used:

Sulphuric acid	1.38%	Magnesia	1.54%
Gravity	3.06	Water and steam tests	O. K.
Initial set	1 hr.	Hard set	4 hr. 30 min.
Residue, No. 200 sieve	27.5%	Residue, No. 100 sieve	8.0%

One-day test	385 lb.
One-week test	716 lb.
One-week test, sand	315 lb.

The concrete was deposited under the supervision of experienced men, who stated that all the piers were treated in the same manner.

Shortly after the timber caisson was removed, Pier 5 was injured between high and low water by a floating barge. This injury was repaired by patching with concrete. Examinations of this pier in 1909 showed that the patch had disappeared entirely, and that the concrete

Mr. Yates.

was being eaten away gradually between high and low water at the point of injury. All the remaining piers except two showed some deterioration of the concrete between high and low water. In the case of Pier 5 the concrete was eaten away to a depth of 20 in. Two piers showed practically no deterioration. An examination by a diver in 1909 showed that there was no deterioration of any of the piers below a point about 1 ft. below mean low water. From observations, the piers most affected were those which had the greatest exposure to floating ice or drift.

Attempts were made to get samples of the disintegrated concrete for the purpose of making a chemical analysis to determine, if practicable, the action of the sea water. This was found impracticable, but when the samples were first removed and still wet, a large number of crystals of a white salt were visible, and these seemed to disappear entirely as the concrete dried out.

These piers have been repaired several times—once with a cement gun, which latter repair lasted about 4 years. The repairs have somewhat retarded the disintegration of the concrete, but the destructive action has been found to be practically continuous. It is very noticeable that the action is not general over the whole surface, but is rather confined to spots, such as the injured section of Pier 5.

Another example may be cited, not particularly for the disintegrating action of the sea water, but rather on account of the fact that a sample of the white salt, mentioned in reference to the Newark Bay Bridge, was obtained and analyzed; this is the bridge of The Central Railroad Company of New Jersey over the Shrewsbury River about a mile from its outlet in Sandy Hook Bay, and built in 1913. Ten piers were constructed in sea water, and of these, two showed signs of failure about a year after they were built. These piers were rebuilt, and an investigation into the cause of the failure developed the fact that, in both of them, the concrete of the foundations, which had been deposited in water with a tremie, had been deposited at low tide, when the chute to the tremie was very flat. The concrete mixer had been on a floating barge, and the low tide had caused the flat condition of the chute which resulted in the concrete being mixed very wet and the tremie not being kept full of concrete during the process of depositing.

The concrete foundations of these piers failed completely, and when removed showed stratification, consisting of alternate layers of a putty-like substance and of sand and gravel with a little cement, or rather, a very weak mixture of concrete. Samples of this mixture were taken, and the small crystals of white salt were plainly visible when the concrete was first removed and before it had time to dry out. Enough of this white powder was collected, and an analysis was made

Mr. Yates. by Mr. C. M. Chapman, Engineer of Tests, of Westinghouse, Church, Kerr and Company, a copy of which is as follows:

"We have made an analysis of the sample of white powder which was scraped from the sample of disintegrated concrete from the Shrewsbury River bridge pier, with the following results:

"Free Silica	$Si O_2$	57.64%
Alumina	$\left\{ \begin{matrix} Fe_2 O_3 \\ Al_2 O_3 \end{matrix} \right\}$	7.44
Calcium	Ca	6.83
Magnesium	Mg	8.66
Chlorides	Cl	1.82

"The large amount of silicia is undoubtedly due to the grains of sand which were scraped off the concrete with the sample. Eliminating the silica from consideration, we find that the sample is made up of approximately equal parts of magnesium, calcium, and iron alumina oxides. The magnesium is somewhat in excess of the calcium, which indicates a transfer of the calcium and magnesium bases between the cement in the concrete and the salt in the sea water. The magnesium of the sea water replaces the calcium of the cement. This action can only take place where the sea water has free access to the cement, or, in other words, at points where the concrete is quite porous. In a dense concrete such action could not take place, or would be so slow that it would be almost negligible."

At the south or up-stream end of one of these piers, when examined about 6 months ago, there was disintegration over a small area to a depth of about 2 to 3 in. between high and low water. This end of the pier is very much exposed to damage from floating ice.

Another example of the deterioration of concrete in sea water is one of the piers of the bridge of The Central Railroad Company of New Jersey over the Elizabeth River at Elizabethport, N. J. Due to delays which were beyond control, it was necessary to deposit the concrete for these piers in freezing weather. Parts of the surfaces of the piers were injured slightly by freezing, which resulted later in a peeling of the surfaces. These surfaces were patched with a cement gun, and have shown no further peeling, with the exception that the patching on the down-stream end of one of the piers has fallen off. Where the injured surface of this pier is between high and low water the concrete has disintegrated to a depth of from 4 to 6 in.

The water of this stream is salt, but is apt to carry a large mixture of sewage and fresh water. The pier surfaces are covered with slime between high and low water, but, with the exception noted, show no deterioration such as might result from the action of salt water.

In regard to the construction of the New Hackensack and Passaic River Bridges by The Central Railroad Company of New Jersey in 1912-14, it may be of interest to note that, due to experience with

the disintegration of concrete in sea water, it was decided to build those portions of the piers between high and low water with a granite facing, backed up by a concrete interior. This granite facing was carried from a point 2 ft. below mean low water to a point 2 ft. above high water. Mr. Yates.

Practically the whole of the 12 000 cu. yd. of concrete in the foundation of these piers was deposited in water with drop-bottom buckets. The channel piers extended to a depth of 30 ft. below low water, and the concrete was deposited continuously within timber coffer-dams up to a point 2 ft. below low water. About a day after the completion of the concrete, the water was pumped down, the laitance was removed from the upper surface, and the construction of the remaining portions of the piers was carried on in the air.

The concrete was composed of 1 part of Portland cement, 2 parts of washed Cow Bay sand, and 4 parts of washed and graded gravel.

In experimenting with different concrete aggregates, it was found that when Cow Bay sand and gravel were used, 2 to 3 in. of laitance would result in depositing a depth of 15 ft. of concrete under water. When Cow Bay sand and 1½-in. broken trap rock (dust screened out) were used, the result was about double that quantity of laitance. For this reason practically all the work was done with the gravel.

Examinations of the under-water surfaces by a diver, shortly after the completion of the structure, developed no defects, except that on one of the piers there was found a wedge-shaped horizontal band of laitance, from 4 to 6 in. wide, a section of which the diver was able to remove to a depth of about 4 in. Investigations showed that this band of laitance was due to several days' stoppage of the work of depositing the concrete at this level, and that, although attempts had been made to remove the laitance with a clam-shell bucket and the help of a diver before the depositing was again started, the efforts were not entirely successful.

Annual general inspections of all the structures on the Central Railroad are made by the Bridge Engineer, and no disintegration similar to that found in salt water has ever been observed in concrete piers in fresh-water streams, although in many cases the concrete is exposed to damage from floating ice. Concrete surfaces have been found injured (probably by floating ice), but no disintegration has taken place. This rather disproves the theory that the disintegration of concrete in sea water, between tides, is due solely to the alternate freezing and thawing of the concrete.

The result of the speaker's experiences with the structures cited leads him to believe that concrete can be used safely in sea water, provided there is no danger of the surfaces being injured by floating material. Where such action is at all likely to take place, it has been

Mr. Yates. decided to use a stone facing backed up with concrete. The speaker has no hesitation in using properly deposited concrete to a point 2 ft. below low tide, or to a height that is below any likelihood of injury by floating material.

Mr. McClintock. J. R. McClintock,* M. Am. Soc. C. E.—The speaker had an experience, at Clarksburg, W. Va., with disintegration of some concrete beams across the top of an open coagulating basin. These beams are normally above the water level, but at times, owing to careless operation of the pumps, they are alternately submerged and exposed, causing in cold weather very severe freezing conditions.

The basin was constructed 7 years ago, and after about 5 years probably two-thirds of these beams were almost completely disintegrated, the concrete looking practically like a loose shale. The other beams, although under the same conditions, were practically intact.

As far as known, there was no difference in construction methods. They were all built by the same gang, with the same foreman, and presumably of the same materials. It is possible, however, that in some of these beams use was made of a local stone which is subject to disintegration when exposed to the weather. There were some delays in the shipment of the hard limestone regularly used, and it is very likely that some of the poor local stone may have been used, in order to avoid waiting.

The contractor on this work had rather a disappointing experience with the local stone. In excavating for a reservoir, probably 4 000 or 5 000 cu. yd. of stone were piled up to be used later for concrete. After this material had remained piled up nearly a year, it was found to have weathered to almost a soft clay. When taken out it appeared to be a good hard stone.

The other concrete structures at Clarksburg, including a filter plant and a 1 000 000-gal. concrete tank, were substantially impervious to water, as judged by leakage measurements, and it is puzzling to understand just why the beams proved to be so porous as to allow the entrance of water to cause their disintegration.

There is another factor which should be mentioned. The river water receives a great deal of acid mine waste, and the sulphur compounds may have had some effect on the concrete. This, however, would not explain a selective action on some parts and not on others. Probably the best explanation is that poor stone was used in a portion of the beams, and good stone in the rest. The stone was of a nature between a shale and a sandstone. The different strata change gradually from a true shale into a sort of stone locally called sandstone.

* New York City.

S. B. Williamson,* M. Am. Soc. C. E.—Before the construction of the locks on the Panama Canal, there were several independent investigations as to the effect of sea water on concrete, and all the information obtainable was gathered at that time. According to the speaker's memory, the failures recorded were due largely to porous or pervious concrete. On the strength of this information, the engineering authorities on the canal agreed to use concrete in the lock construction; and in the lower parts of the locks that were subject to sea water a richer mixture was used. Mr. Williamson.

At Miraflores, where there is a 20-ft. tide, and the lock gates are about 83 ft. high, an interesting condition developed: It was found that the difference in the weight of the salt water on the lower side and the fresh water on the inside of the gates made it possible to have a head of about 18 in. on the lower side of the gates, tending to force them open when the lock was full.

In order to discover a means of overcoming this, a number of experiments were carried out, and it was finally ascertained that, by turning up the lower ends of the culverts so as to discharge at a certain elevation, the inter-mixture or diffusion took place in such a way as to equalize the pressure heads.

Thus far, the concrete in the locks has shown no signs of disintegration. There has been some disintegration, however, in the concrete docks built at Balboa. These are reinforced docks, supported by reinforced concrete cylinders, and some of the floor girders are partly submerged at extreme high tide. These show some disintegration, especially where cracks have appeared, but as yet there is nothing serious.

Great care was taken in the composition and placing of the concrete in the locks, particularly where they will be subject to sea water, and the speaker believes that the concrete is impervious and will not be readily attacked.

Waldo C. Briggs,† M. Am. Soc. C. E.—There is one point which the speaker would very much like to have developed. If "the addition of clay to the cement had a slightly beneficial result" with the concrete subjected to such unusual conditions, why not modify the customary specification for sand, requiring it to be "clean and sharp", to read "clean and sharp, free from dirt, loam, mica, and organic matter, and shall contain not more than 5% by volume of clay"?‡ Mr. Briggs.

It happens sometimes that what appears to be a very good grade of sand is encountered in excavation made in connection with construction work; but if this sand contains clay, it necessarily is rejected if the specifications require it to be clean.

* New York City.

† Long Island City, N. Y.

‡ Bureau of Sewers, New York City.

Mr. Bilyeu. CHARLES S. BILYEU,* ASSOC. M. AM. SOC. C. E.—A curve of maximum density, or an ideal curve for the combination of sand and aggregate, has been plotted from the results of an extended series of tests by William B. Fuller and Sanford E. Thompson, Members, Am. Soc. C. E., at Jerome Park Reservoir, New York City, in 1902-04.

The speaker's criticism of specifications refers particularly to those for reinforced concrete buildings. It is true that such specifications are frequently prepared in architects' offices, and that engineers are not responsible for them. They generally cover adequately the testing of the cement, but rarely contain any requirement for the analysis of the sand and aggregate.

The speaker has been called on, from time to time, to make tests of materials (particularly sand) used in large and important buildings. In several instances the sand has been found to contain an abnormally large percentage of very fine grains, and to be otherwise unsuitable for use in work of such character. Tests of this nature are usually arranged for only after the work is well under way and after it has been found that the concrete has not developed proper strength.

In the speaker's opinion, careful sand and aggregate analyses are very important investigations, and are absolutely necessary for safe and economical concrete construction.

Mr. Day. W. E. DAY,* ASSOC. M. AM. SOC. C. E.—In line with the discussion of sand mixtures, an incident on some construction work during the past season might be described. A sand pit was opened, about 2½ miles from the work, which contained a sand of excellent quality. The testing laboratory reported that it was very clean, and that the percentages of various sizes were such as to make it an almost perfect sand for concrete.

On account of the long haul, the experiment was made of mixing this pit sand with a very fine beach sand, containing considerable loam, which was right on the works. Test cubes and briquettes were made with pit sand containing from 0% to 45% of beach sand. These cubes and briquettes were tested after being stored 30 days, and the results showed that adding up to 30% of beach sand improved the strength of the concrete, as compared with concrete made from the so-called perfect sand.

Mr. Ridgway. ROBERT RIDGWAY,* M. AM. SOC. C. E.—It is generally agreed that the grading of sand is very important. A well-graded sand containing a small quantity of clay is preferable to a poorly-graded sand which is free from impurities, particularly if the sand is uniformly fine. It is well, however, to reduce the quantity of clay and other impurities to a

* New York City.

minimum, and what one should strive to get is a clean, well-graded sand. The speaker does not agree with those who have advocated the use of a small quantity of clay to increase the density of the concrete. It is better to omit the clay and substitute a little more cement, in order to secure this density, thereby increasing the strength of the concrete as well. If the clay appears as lumps in the sand, it is obviously more objectionable, as Mr. Fuller states, than if in the form of a powder. The holes often seen in the surfaces of cement walks and in the faces of concrete walls are generally caused by the dissolving of lumps of clay which were incorporated in the mix. Mr. Ridgway.

Mention has been made of limestones as concrete aggregates. The use of limestones was not permitted in the Catskill Aqueduct specifications for pressure-tunnel lining because of their solubility. This was the result of a careful study of the subject by the Designing Division of the Board of Water Supply before the specifications were prepared. It was found, in a number of instances, that concrete had disintegrated, due to the solubility of the coarse aggregate. A notable example cited was that of the Thirlmere Aqueduct for the City of Manchester, England. In this case, as the speaker understands it, an almost pure limestone was used, and the soft water in the Aqueduct dissolved it to such an extent that the concrete was badly disintegrated. In most works, however, limestones are not prohibited; for example, the Public Service Commission specifications for subway construction in New York City permit the use of "Sound, hard, broken limestone."

In driving the new rapid transit tunnel under the East River from Old Slip, Manhattan, last year, the shield working under compressed air encountered the concrete foundation of the wall at the river bulkhead line. This concrete had been deposited in bags under the salt water by the Department of Docks and Ferries of New York City about 10 years before. The mix was said to be 1:2:5, and it was found to be in excellent condition, as sound and dense as though it had been placed behind forms in the dry. The burlap of the bags was still there, but the concrete was bonded so well that it looked like good mass concrete. There was no sign of any disintegration.

Mr. Fuller.

GEORGE W. FULLER,* M. AM. SOC. C. E.—This discussion brings out four leading points: First, that imperviousness of concrete is a *sine qua non* for structures subjected to a fluctuating water line; secondly, laitance must be carefully guarded against; thirdly, there is the question of temperatures, in relation to disintegration by alternate freezing and thawing; and, fourthly, there is the question of what may be brought about by organic matters and their decomposition to form acids under putrefactive processes.

* New York City.

Mr. Fuller.

Sanitary engineers know well, in connection with water-works and sewerage structures, what porous concrete may signify, especially as to the stability and integrity of structures above and near the flow line, as ordinarily seen.

Mr. McClintock has given some interesting observations on concrete sedimentation basins of the water-works plant at Clarksburg, W. Va. This shows what freezing and thawing may do, in the absence of any acids or organic matter to decompose the structures, where concrete, probably not as impervious as it should be, is subject to the various influences of the elements, especially where there is a fluctuating flow line.

Now for a few words in respect to the fourth element, and that is the disintegration of concrete, mortar, or brick, under those circumstances where the decomposition of organic matters of sewage or polluted water may bring about substances which produce acids.

That is probably an old story. It has been debated, year in and year out, in relation to the question as to whether concrete is as good as brick for large sewers.

It is a fact that the organic matters contained in sewage under some circumstances will undergo bacterial decomposition, and that the sulphur compounds in the organic matters will be changed to sulphureted hydrogen, in the complete absence of dissolved atmospheric oxygen, and in the presence of the right kinds of bacteria which ordinarily grow there. It is a further fact that, when bacterial agencies bring about what are commonly called putrefactive changes, sulphureted hydrogen and other products of putrefaction may be obtained from mineralized sulphur compounds, such as sulphate of lime or magnesia, and found in cement, fresh water, and sea water.

Sea water is so highly mineralized that we probably do not realize what opportunities there are for the formation of compounds of a dissolving nature. Neither do we know what inter-reactions may produce, in conjunction with putrefaction, as to solvents from magnesium chloride and other neutral salts of a supposedly stable and non-corrosive nature with respect to concrete.

The formation of sulphureted hydrogen does not cause an appreciable corrosion of fairly good concrete below the minimum water line of a sewer, septic tank, or pier of concrete, so far as the speaker's observations and studies go. Even in septic tanks, he has heard of only one instance of serious disintegration of concrete in the lower part of the structure, and that was in the case of a tank in the arid regions of the Southwest.

Sulphureted hydrogen must be converted into sulphuric acid in order to develop corrosive properties. This is effected when the gaseous sulphureted hydrogen rises above the water, meets the oxygen of the

air, and in the presence of moisture. This is why sewers, septic tanks, and piers are disintegrated above and not below the flow line. Of course, this solvent action varies in intensity with the porosity of the concrete, effect of freezing, etc. Mr. Fuller.

The outfall sewer of Los Angeles showed disintegration of mortar and signs of deterioration of brick in manholes beyond the main siphon, but not below the flow line. When the atmosphere with its oxygen was excluded, this disintegration was stopped to a surprisingly satisfactory extent.

Much sulphureted hydrogen does not escape into the atmosphere, as it combines with iron salts to form the black ferrous sulphide of iron, which gives liquids in process of putrefaction their black characteristic appearance. This is why the sludge of septic tanks is black, and why the sediment in pump wells and settling basins is black, as was reported in detail by the speaker some 20 years ago in his water purification investigations at Cincinnati.

Laitance should certainly be kept within reasonable limits, otherwise undesired results are to be expected. Undoubtedly, the pendulum is swinging back from the very wet mixes of recent years toward the dry mixes of 20 years ago. However, the mix should not be so dry as to make it problematical as to removing entrained air from the pores of the concrete.

It is feasible to use sand with a moderate quantity of silt or clay without vitiating the integrity of a structure, if that fine material can actually be used as filling uniformly a certain portion of all voids of the sand as a substitute for cement; but if such silt or clay is in lumps or is otherwise irregularly and improperly dispersed in the voids of the sand, the programme becomes invalid.

Fine material is objectionable if it occurs as a coating on the sand particles, as it weakens the concrete; or if it contains sufficient organic matter to interfere with the setting of the concrete; or if it is flaky or of such character as to produce lumps, on mixing the concrete.

On the other hand, fine material may add materially to the imperviousness and density of the concrete. This is a function of the mix of concrete, the nature and quantity of the fine material; the percentage of voids in the sand; and the relative size and shape of the sand particles.

As Mr. Ridgway says, clean sand filled with an excess of cement has marked advantages in the direction of safety; but this is not always practicable in some localities, or worth its cost in others—as compared with using sand with fine material in it.

At York, Pa., the local building sand is a crushed rock containing 10% or more of fine material passing a 100-mesh sieve. It was used with satisfaction on several hundred thousand dollars worth of work

Mr. Fuller. on outfall sewers, pumping station, and sewage treatment plant. Apparently, this material did not form lumps. Its use saved the needless added cost of obtaining other sand transported from a long distance. However, this is a procedure which ought not to be adopted without careful investigation in each local case.

Mr. Rhett. A. H. RHETT,* ASSOC. M. AM. SOC. C. E. (by letter).—The subject of the disintegrating action of sea water on concrete has received much discussion among engineers, and the results of the tests at the Boston Navy Yard, as described by Mr. Bakenhus, seem to the writer to throw some light on the subject and point to one primary and basic cause of disintegration. This is the porosity of the concrete.

In *Technologic Paper No. 12*, of the Bureau of Standards, on the "Action of the Salts in Alkali Water and Sea Water on Cements", it is stated that "a hydraulic cement is readily decomposed if intimately exposed to the chemical action of various sulphate and chloride solutions", if these solutions are of sufficient strength.

The most conspicuous result of these tests is, that whatever disintegration did occur, occurred approximately between high and low water. It is quite obvious that, if the concrete is porous, the pores would be filled with water at high tide, that the salts of the water would be deposited by evaporation during the recession of the tide, and that this process would be repeated until the solution in the pores would become of sufficient strength to give rise to this chemical action. It must be, therefore, that if a concrete is porous, it is subjected to a much stronger chemical action between high and low water than elsewhere, due to this accumulative concentration; on the other hand, if it is non-porous, no such concentration can occur. It is true that evaporation is necessary for the precipitation of the crystals the increase of which in volume causes the actual disruption or disintegration of the concrete, but the unaffected pieces were subjected to the same alternate wetting and drying as the affected ones, and this alone cannot account for the erosion. It seems possible, also, that sea water in its normal condition is not sufficiently strong in sulphates and chlorides to produce chemical action.

The writer has re-stated the conclusions drawn by Mr. Bakenhus on the results of the tests, which conclusions are substantiated by a detailed examination of those tests, and it will be seen that in each case these conclusions can be interpreted as confirmatory of the proposition that the disintegration was more or less proportional to the porosity of the concrete.

"(*a*) That the 1:1:2 mixture is superior to the $1:2\frac{1}{2}:4\frac{1}{2}$, and that the $1:2\frac{1}{2}:4\frac{1}{2}$ is, in turn, superior to the 1:3:6."

* New York City.

Mr. Rhett.

It requires no exposition to confirm the statement that the richer the mix the less porous the concrete, with similar conditions of mixing and placing.

"(*b*) That the wet mixtures are superior to the dry."

The tests made by the Bureau of Standards on the hydration of Portland cement, as reported in *Technologic Paper No. 43,* show that there is a latent element which in many cements does not begin to hydrate until 28 days or later, but if then continued in the presence of water, it hydrates with a large increase in volume and with an amorphous structure. It is obvious that this action tends to close the pores of the concrete by filling them with this amorphous material, and that this inert, but most important, action cannot occur except in a wet concrete which is not permitted to dry out too quickly. In the present instance, it was specified that the forms should be "matched" and "as tight as possible" so that the water in the wet mix was retained.

In *Technologic Paper No. 58,* of the Bureau of Standards, on the "Strength and Other Properties of Concrete as Affected by Materials and Methods of Preparation", it is shown from the tests made that, with a given mix, the resulting compression strength of the concrete was about 100% greater with 10.5% of mixing water than with 6.5%; that it was about 25% greater when cured in a moist atmosphere than in the open air; and that the compressive strength is proportional to the density.

"(*c*) That the effects of magnesia and alumina in varying proportions are not very marked, and follow no apparent law, although the two most durable specimens are those lowest in alumina content."

This conclusion is confirmed by *Technologic Paper No. 12,* previously referred to, which states that "contrary to the opinion of many, there is no apparent relation between the chemical composition of a cement and the rapidity with which it reacts with sea water when brought into intimate contact"; and that "the quantity of alumina, iron, or silica present in the cement does not affect its solubility."

"(*d*) That extra care in mixing produced decidedly beneficial results."

The experiments of Michaelis, Cabolet, and others have shown that when cement is continuously mixed with water an increase in volume as much as seven hundred-fold may occur. Such an increase in the volume of the cementitious material would tend to fill the natural voids in the aggregates and make the concrete non-porous.

"(*e*) That hydrated lime was of no benefit, but rather a detriment."

To refer again to *Technologic Paper No. 12,* it is shown that lime is the element of the cement that is attacked by the sulphates and

Mr. Rhett. chlorides of the salt water. This action is shown to be even more pronounced with set than with unset cements, the reason for this being that free lime is always liberated in the process of setting. The addition of more lime to the cement of course will increase this action. In this specimen, too, the 10% of lime replaced an equal quantity of cement, and decreased to this degree any possible automatic pore-closing effect from the full hydration of the cement as already described.

"(*f*) That the addition of Sylvester wash was harmful."

The theory of the addition of alum and soap is that they combine and coat the pores with a water-repellant material. It is difficult to see, however, how such an action could occur, and yet the particles of the cement remain uncoated with the same water-repellant material which would prevent their full hydration and the swelling and void-filling action resulting therefrom.

"(*g*) That the addition of clay to the cement had a slightly beneficial result."

It is well known that clay is a colloidal material with respect to water, that is, that it remains suspended in water or holds water in suspension. When mixed with cement, it would seem that this action could not but retard evaporation, and then tend to promote the further hydration of the cement. It is also probable that the clay would have some void-filling effect.

It is affirmed, quite confidently, sometimes, that this disintegration of concrete between high and low water occurs only in northern latitudes, though the writer has never seen any very convincing evidence adduced to substantiate this claim, and that the disruption of concrete is due to the freezing of water in the pores and not to crystallization, neither action can occur, however, without the open pore to pocket the water, and to the open pore must be attributed the primary cause of trouble.

Mr. Larsen's discussion (pages 693-699) reporting his tests on the disintegration of concrete by salt water is interesting in its general confirmation of the results reported by Mr. Bakenhus as to the no-longer-to-be-questioned fact of such disintegration, but it is impossible to pass his final conclusion unchallenged.

He states that his results "confirm in a general way the conclusion of Mr. Bakenhus that * * * integral water-proofing has a detrimental effect on concrete." The writer has studied Mr. Bakenhus' paper very carefully, and fails to find anything either in his conclusions or in a detailed analysis of the tests to indicate even in the remotest way that "integral water-proofing has a detrimental effect on concrete." That paper states very explicitly that integral water-proofing was used only in two specimens (Nos. 22 and 24) and that both of these were lean mixes (1:3:6). In No. 22, 10% of

hydrated lime was "substituted for an equal quantity of cement;" in No. 24, "to each bag of cement there was added 4¾ lb. of clay." Clay is just as properly an integral water-proofing as lime, unless Mr. Larsen confines his definition of integral water-proofing solely to hydrated lime, and, if he does, his failures are easily explained, as will be shown later. Mr. Bakenhus states that No. 24 "came through * * * better than all the other specimens of a similar mixture, save only No. 15 (low alumina cement) and No. 20 (extra well-mixed concrete)." In other words, thirteen specimens of the 1:3:6 mixture were made, and the one to which the integral water-proofing, clay, had been added, ranked third in quality, the two ahead of it being special. Mr. Rhett.

Now, as to No. 22, to which hydrated lime had been added, Mr. Bakenhus states:

"This specimen is also poorer than eight of the other ten similar specimens in series, showing conclusively, so far as one specimen can, the deleterious effects of hydrated lime when used in concrete immersed in sea water."

This finding is entirely in accord with those of other investigators, and entirely in accord with the now generally accepted theory as to the cause of the disintegration of concrete by sea water, but it happens that, primarily, this has nothing to do with the theory of integral water-proofing.

It has been shown* that the lime is the element in the cement which is attacked by the sulphates and chlorides in the sea water, and this fact has been substantiated by other investigators. The addition, then, of more lime could not fail to increase this action, and, in the specific case of Specimen No. 22, the lime added replaced an equal percentage of cement, and this made a leaner mix.

As to Mr. Larsen's own tests, they certainly do not show that "integral water-proofing has a detrimental effect on concrete." Such a statement could only mean either that all integral compounds were the same, or that he had tested all known compounds, and there is certainly nothing shown in his tests to substantiate either claim. As a matter of fact, there are many of these materials on the market. The Bureau of Standards has reported† tests on more than twenty, and the theories on which they are based are radically different.

In fact, the hazy comprehension with which the subject of integral water-proofing continues to be discussed is astonishing, in view of the simplicity of the rational theory on the subject, and seems to have its foundation in a very general lack of knowledge of some of the most important characteristics of the most widely used of engineering materials, namely, Portland cement. Portland cement seems to con-

* *Technologic Paper No. 12*, Bureau of Standards.

† *Technologic Paper No. 3.*

Mr. Rhett. tinue to be regarded as a more or less passive agent, whereas the experiments of Michaelis, Cabolet, the Bureau of Standards, Nathan Johnson, and others have shown, on the contrary, that it contains elements which, though latent under ordinary conditions, are capable, under proper conditions, of a great increase in volume during hydration. The function which this characteristic may be made to play in rendering concrete non-porous seems to be complacently overlooked. It is possible to bring this action into play with a very great void-closing effect, and it is possible, moreover, to reduce the voids by lubrication of the aggregate; and the accomplishment of these two perfectly comprehensible results is the rational theory of integral water-proofing.

The elimination of the pores is acknowledged and has been proved to be the secret of the preservation of concrete subject to salt water tidal influences, and any elimination based on these two simple principles would be, and has been proved to be, just as effective under salt water as elsewhere. There are two conspicuous instances in the vicinity of New York City where integral water-proofing has been used in concrete immersed in salt water, and the concrete has been unaffected. These are Dry Dock No. 4 of the Brooklyn Navy Yard and the pier on Mr. J. P. Morgan's place at Glen Cove, Long Island.

Mr. Brown. Marshall W. Brown,* Assoc. M. Am. Soc. C. E.— The discussion of this paper has drifted toward the failure of concrete structures, due to the action of sea water. The speaker believes that great attention should be given to the nature of the stone used in the concrete.

If it is assumed that a structure has been adequately designed and properly built to withstand the forces acting on it as a structure, then its life depends mainly on its ability to resist the constant and all-powerful force of sea water acting on it as a destructive agent.

This force is both chemical and mechanical. The chemical reaction, being promoted by the presence of moisture, is facilitated by the mechanical action of the water, which, carrying away particles that have been disintegrated, presents new surfaces to be acted on.

The chemical destruction of any particular mass of concrete is slow. The rate of the mechanical destruction varies greatly, depending on the intensity of the wave action and the nature of the surface on which that action takes place.

The rate at which concrete disintegrates increases with the intensity of the wave action, and with the increasing roughness of the surfaces exposed.

In any volume of concrete there are three materials which are acted on, namely, the cement, the sand, and the stone. As the cement which binds the whole mass together is the one material which is

* New York City.

most readily acted on chemically, its rate of destruction must be reduced to a minimum. This is done in a number of ways: Mr. Brown.

First.—By reducing the quantity of cement needed in a mass, that is, by proportioning the sizes of the sand and stone in such a way that the volume of the voids filled by the cement in the mass is as small as possible.

Second.—By using sand and stone which are in themselves chemically inert and impervious to moisture, and by placing the mixture of these materials in such a manner that the mass, as a whole, when set, is impervious to moisture.

This eliminates all chance of disintegration of the interior of the mass, and confines it to the surface exposed directly to the action of the water. Without moisture, chemical action cannot take place.

The first disintegration of the surface of concrete takes place very slowly, and is mainly chemical. The surface is hard and smooth, and the water has slight effect on it mechanically. Gradually, the cement is eaten away, and a close examination shows that the area of chemical action has been greatly increased around the particles of sand. These are washed away, and, as the action continues, the stone is exposed. After a time, the whole surface, exposed to the action of the water, appears to be one mass of stone surrounded by small seams of mortar composed of sand and cement.

Thus far, only the outer surface of the structure has been displaced, and the integrity of the whole structure thereafter depends mainly on the character of the stone used. On it comes the chemical action of the sea water, the intensive mechanical action of the waves dashing against it, as well as the effect of frost, all tending to loosen and tear each individual particle of stone from the cement which binds it to the structure.

For this reason the speaker believes that the character of the stone has much to do with the life of a concrete structure exposed to the action of sea water. The stone in the concrete should be chemically inert, insoluble, non-absorbent, and should present a surface to which the cement will adhere with the greatest possible strength.

As far as local structures are concerned, this would practically limit such stone to crushed trap rock, granite, or washed quartz gravel.

The speaker has seen many failures of concrete structures in sea water, and in each instance he believes it has been due to the use of shale rock, sandstone, limestone, and other rocks of a sedimentary nature.

Albert Larsen,* Assoc. M. Am. Soc. C. E. (by letter).—In. 1912 the Grand Trunk Railway System reconstructed a portion of the Mr. Larsen.

* Providence, R. I.

Mr. Larsen.

Back Cove Bridge, at Portland, Me., building two rest piers and re-capping the center pier. The center pier, built in 1892, was of granite ashlar, and was in such good condition that it is still used as the center pier, even with the increased loading. During the progress of the construction of the piers in 1912 Mr. Armour, Masonry Engineer, Mr. Durham, of the J. S. Metcalf Company, and the writer investigated the durability of concrete in sea water in order to determine its suitability for the new rest piers. Various concrete structures (including the test piles of the Aberthaw Construction Company), were visited in Boston Harbor, and from these observations it was finally decided to face the rest piers with granite ashlar, backed with concrete. The Grand Trunk Railway System contemplated future work in Portland Harbor, and therefore actual results of concrete placed in sea water were desirable. Ten 12 by 16-in. test piles, 16 ft. long, were made by various water-proofing methods, and immersed in sea water. Six piles were treated with integral water-proofing; two were plain, one of which was cured in the open for 3 months before being immersed; and two were treated with external water-proofing. A 1:2:4 mixture was used, with sand and crushed stone aggregates. The percentage of voids in the sand was 33.

The proportions and different methods of water-proofing for the piles are stated in Table 11.

TABLE 11.

Pile No.	Treatment.	Cubic feet of cement.	Cubic feet of stone.	Cubic feet of sand.	Gallons of water.
1....	Plain (cured in open, 3 months)	5	20	10	25
2....	Lye and alum (external)	5	20	10	25
3....	Plain	5	20	10	25
4....	Water glass (external)	5	20	10	25
5....	Water-proof cement ("*A*")	5	20	10	25
6....	Regular cement plus the water-proofing compound ("*B*")	5	20	10	25
7....	Regular cement plus the water-proofing compound ("*C*")	5	20	10	29
8....	Regular cement plus the water-proofing compound ("*D*")	5	20	10	24
9....	Regular cement plus the water-proofing compound ("*E*")	5	20	10	23
10....	Regular cement plus the water-proofing compound ("*F*")	5	20	10	25

Water-proof cement was used in Pile No. 5, and the others contained "Knickerbocker" cement, either alone or in combination with water-proofing compound. A ½-yd. Smith mixer was used; each batch was turned 26 times, and the time was 1½ min. Portland City water was used.

Mr. Larsen.

The following results were obtained with the cement:

	Neat.	Sand.
One-day test	390 lb.	...
Seven-day test	628 "	318 lb.
Twenty-eight-day test	735 "	405 "

The chemical analysis of the cement was as follows:

SiO_2	23.46
Fe_2O_3	9.35
CaO	61.24
MgO	3.59
SO_3	1.60
Loss	0.64

The piles were fastened to the pile trestle adjoining the spur-track leading to the coal pockets, and were made and immersed in the water on the following dates:

	Moulded.	Placed.
No. 1	October 10th, 1912.	January 23d, 1913.
2	" 3d, "	October 23d, 1912.
3	" 10th, "	" 24th, "
4	" 3d, "	" 24th, "
5	" 3d, "	" 18th, "
6	" 10th, "	" 24th, "
7	" 4th, "	" 18th, "
8	" 4th, "	" 18th, "
9	" 4th, "	" 18th, "
10	" 4th, "	" 18th, "

The following is the formula for the Sylvester wash used in Pile No. 2: To 2 gal. of water add 1 lb. of concentrated lye and 5 lb. of alum, and mix until completely dissolved. This is a concentrated stock. When used, 1 pint of stock solution and 10 lb. of cement are mixed with enough water to make a mixture that will lather freely under the brush. Two coats are applied, the first as soon as the forms are removed (the surface must be wet), and the second as soon as the first is dry.

The formula for the water-glass or sodium silicate method of water-proofing (Albert Meyer) is as follows: After the forms are removed, grind with a carborundum stone any projections due to the concrete seeping through the joints between the boards. Keep the surface damp for 2 weeks after placing the concrete. Wash the surface thoroughly and allow it to dry. Mix a solution of 1 part water-glass (sodium silicate) 40° Baumé, with from 4 to 6 parts of water, total 5 to 7 parts, according to the density of the concrete surface treated.

Mr. Larsen. The denser the surface the weaker should be the solution. Apply the water-glass solution with a brush. After 4 hours and within 24 hours, wash off the surface with clear water. Again allow the surface to dry. When dry apply another coat of water-glass solution. After 4 hours and within 24 hours, again wash off the surface with clear water and allow to dry. Repeat this process for three or four coats, which should be sufficient to close the pores.

The water-glass which penetrates the pores comes in contact with the alkalies in the cement and concrete, and forms an insoluble hard material, causing the surface to become very hard to a depth of $\frac{1}{8}$ to $\frac{1}{2}$ in. according to the density of the concrete. The excess of sodium silicate which remains on the surface, not having come in contact with the alkalies, is soluble, and is easily washed off with water. The reason for washing off the surface between each coat and allowing the surface to dry is to obtain a more thorough penetration of the sodium silicate.

The tops of the concrete piers were treated by the water-glass method, and when examined on March 13th, 1917, were found to be in as good condition as when treated. The tops were finished with a 1:2 mortar with granite chips.

The piles were examined by the writer on March 13th, 1917, and photographs then taken are reproduced as Figs. 2 and 3.

The fungous growth was not removed from the piles.

The following indicates the condition of the piles:

No. 1 pile: Perfect condition.
No. 3 " : " "
No. 4 " : Nearly perfect condition.
No. 2 " : " " "
No. 5 " : Very marked deterioration at base of pile.
No. 7 " : " " " from low to high tide.
No. 9 " : " " " " " " " "
No. 6 " : Very decided deterioration from low to high tide; very bad condition.
No. 8 " : Very decided deterioration from low to high tide; very bad condition.
No. 10 " : Very decided deterioration from low to high tide; very bad condition.

The results are very interesting, and confirm in a general way the conclusion of Mr. Bakenhus that a 1:2:4 is better than a leaner mixture; that external water-proofing does not help when the concrete has been made as dense as possible; and that integral water-proofing has a detrimental effect on concrete.

From the writer's observations he would say that, once the concrete is affected by the action of the sea water, the rapidity of the disin-

FIG. 2.—SPECIMEN PILES TESTED IN SEA WATER.

FIG. 3.—SPECIMEN PILES TESTED IN SEA WATER.

tegration is increased, as the water gets in between the stones and the mortar faster and thus breaks up the concrete. The writer thinks this may be overcome by facing the concrete with a dense mortar to a depth of from 1½ to 2 in. This may be done during the process of building, or afterward by the cement gun. It would be interesting to experiment with concrete made by either of these methods. Mr. Larsen.

W. Watters Pagon,* M. Am. Soc. C. E. (by letter).—The author has summarized the most valuable series of tests that have been made in the United States; tests that are on a par with the several series made in Germany, Norway, and Russia some years ago; and the writer feels that he should be accorded the sincere appreciation of the Profession. Mr. Pagon.

The destructive action of sea water on concrete has become in recent years a very serious subject in all those cities on the Atlantic Coast north of New York City, and in corresponding climatic regions on other sea coasts. South of New York City, the writer is aware of only two cases of disintegration, one at Atlantic City† and the other in Chesapeake Bay, but both were special cases.

The writer's experience in a specific case on the Connecticut shore of Long Island Sound may be of interest, and is quoted briefly here from a paper written by him, and now on file in the Society's Library. The concrete which suffered from the sea-water action consisted of the copings of a series of bridge piers which were faced with granite masonry below the coping. The facing extended up to mean high tide, but the copings, which extended from this point (Elevation 0) to Elevation + 1.5, were of 1:2:4 concrete without facing.

Pier No. 4, which suffered most severely, was constructed on December 19th, 1913, during weather sufficiently cold to require the use of steam to warm the mixing water. After placing, the concrete was covered with a tarpaulin, stretched from one side of the cofferdam to the other, and cold steam was allowed to escape beneath this. During the week of January 19th to 24th, 1914, there was considerable freezing weather, accompanied by high tides. The result was the formation of a thick ice coating over the concrete surfaces. When this had melted, on January 26th, it was found that the surface of Pier No. 4 had been loosened, and large pieces of scale could be removed. Within a week the mortar between the stones of the gravel had disintegrated and fallen out, leaving the stones projecting. The loose portions were removed with a hammer, and behind them the concrete was perfectly sound. At the end of two weeks, the ice

* Baltimore, Md.

† "Report on the Destructive Action of Sea Water on Concrete", reprinted from the *Monthly Journal* of The Engineers' Club of Baltimore.

Mr. Pagon.

coating formed again, and when this had disappeared the erosion was found to have increased greatly.

The concrete used throughout the work was of a wet "sloppy" consistency. The forms were tight, so that there were no pockets. Immediately after removing the forms, the surfaces were rubbed with carborundum blocks. Cow Bay sand was used, and a mechanical analysis showed it to be too fine; there was also some clay. Analysis of the cement showed it to be excellent. Later, a coarser sand was used, with the results described hereinafter.

In the paper quoted, the writer discussed in detail the various possible causes of this disintegration, and he refers interested readers thereto. He felt that undoubtedly the fineness of the sand was one of the factors causing disintegration, and, therefore, drew up a specification for the sand which was used in all later work, under which, for acceptance, not more than 70% should pass the No. 20 sieve. Long Island sand testing as low as 40% could be obtained from one dealer, but it was found that sand which tested below 60% was too coarse for reinforced concrete work, where a smooth, easy flowing concrete is necessary. To avoid delay in obtaining sand, 70% was adopted as a criterion, and immediately a noticeable change in the strength of the concrete was shown in all the field tests, in the time of setting of the concrete, and also by observation of the concrete itself.

In the writer's opinion, however, the sand was by no means the governing factor. From the detailed description which is given in the paper quoted, it will be seen that the first failure of the surface skin on the concrete always took place where a pebble of the gravel aggregate was close to the surface, and where, therefore, the surface skin had small adhesion to the backing. After this area of skin had peeled off, it was a simple matter for the water, working its way along the surface of the pebble, to force off (by freezing) the surface skin between pebbles. When the skin was gone, the porosity of the concrete, due to the fineness of the sand, caused the disintegration of this mortar, as previously described. Photographs of the concrete just after the beginning of disintegration revealed innumerable little "pock-marks" where the pebbles showed through the broken skin.

In order to test this theory, three specimens were made for the writer by the contractors, McHarg-Barton Company. These were 4 by 4-in. by 10 ft., reinforced axially with a ⅝-in. round bar, hooked at the upper end. The mixtures were as follows:

Specimen	No. 1	1:2:0
"	No. 2	1:2:4
"	No. 3	1:1½:3

These were removed from the forms in 24 hours (in order to simulate as nearly as possible the temperature and greenness conditions of

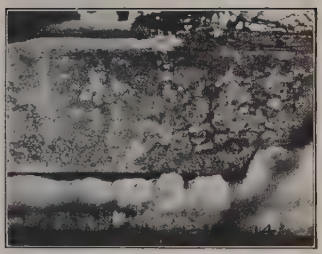


FIG. 4.—PIER NO. 3, MARCH 13TH, 1917. EARLY STAGES, SHOWING PITTING.

FIG. 5.—PIER NO. 4, FEBRUARY 3D, 1914. SCALE GONE; CONCRETE PITTED, SHOWING STONES OF GRAVEL.

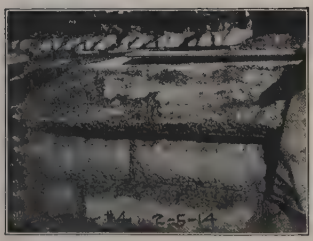
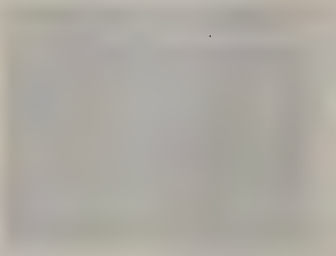
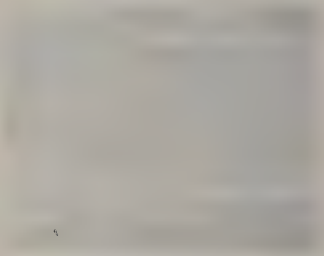


FIG. 6.—PIER NO. 4, FEBRUARY 5TH, 1914. SURFACE SCALE LOOSENED AND FALLING OFF, EDGES ROUNDED OFF BY DISINTEGRATION.

FIG. 7.—PIER NO. 4, APRIL 4TH, 1917. DAMAGED CONCRETE REMOVED; REINFORCEMENT PLACED, READY FOR APPLYING NEW MORTAR FACING.





the pier coping). No. 2 was cracked in handling in one place; No. 3 in three places. When 3 days old the specimens were hung in the river so as to extend above high tide and below low tide. Mr. Pagon.

After the first tide, they were all covered with a white coating, except on the troweled face. At the end of 5 weeks, while they were being shifted to another place, they were accidentally lost. No. 1, however, was in perfect condition; Nos. 2 and 3 were rather badly eaten away, especially at the cracks, where, in fact, No. 2 was eaten away so completely as to expose the rod.

The theory having proved satisfactory, so far as the test went, it was decided to cut away the face of the coping and replace it with 1:2 mortar made of sand bought under the new specification. This was done on April 4th, 1914, the mortar being mixed to such a consistency that it could be poured and stirred like thick cream. The forms were left on for a couple of weeks to protect the concrete from the sea water as far as possible. When they were removed, the mortar was in good condition, although there were numerous instances of bubbles having formed while it was being handled and placed. The surface was not rubbed for fear of causing injury to the protecting skin.

When examined 2 years later, this mortar facing was in perfect condition, although it had passed through two winters. This would seem to confirm the theory, in so far as one test can confirm it.

If other members of the Society have had similar experiences, it would certainly be valuable to hear from them, and the writer's only regret has been that in the Aberthaw tests, described by Mr. Bakenhus, none of the specimens was of mortar only. The writer feels strongly that the relatively good condition of the 1:1:2 specimens is due not only to the greater percentage of cement to aggregate, but also to the greater percentage of cement and sand combined to stone, or, in other words, that a 1:1:2 mixture approaches more nearly to a mortar.

R. J. Wig* and Lewis R. Ferguson,† Assoc. Members, Am. Soc. C. E. (by letter).—The increasing necessity for marine structures, due largely to the improvement of harbor facilities and the military work along our sea coasts, gives an added value at the present time to the paper presented by Mr. Bakenhus. Messrs. Wig and Ferguson.

The points he brings out are largely confirmed by an extensive investigation recently completed by the writers. During this investigation practically all the concrete structures along the Pacific, Atlantic, and Gulf Coasts, as well as a number in Canada and Cuba, were carefully examined. Their present condition was noted, and in most

* Washington, D. C.

† Philadelphia, Pa.

Messrs. Wig and Ferguson.

cases a detailed description of the materials and methods used in construction were obtained.

The effect of the quantity of water used in mixing the concrete is rightly emphasized by Mr. Bakenhus, for this factor undoubtedly has a most important bearing on the ability of the structure to resist sea-water action. He states that the specimens made of wet or very wet mixtures are in much better condition than those made of dry concrete, and this is substantiated by the data he presents.

Perhaps it would have been better if the terms "wet" and "very wet" had not been chosen for describing the consistencies used. Ordinarily, in present practice, concrete which would be designated as "wet" or "very wet", contains a great deal more water than was used in any of the mixtures from which the specimens were made. Even in the very wet mixtures, the notes in Table 3-*A* indicate that the concrete was tamped. In ordinary practice, very wet or even wet concrete would flow into place, and it would be so soft that tamping would not be necessary, nor could it be compacted in this manner. The writers believe, as a result of their investigation previously referred to, that, with concrete subjected to sea-water action, the best results are invariably obtained when a consistency is used which will permit of light tamping. This light tamping should produce quaking in the mass. If the concrete is mixed too dry, a porous condition will result; and if too much water is used in mixing, a lack of density, together with a chalky condition of the surface, results. In both cases, the concrete is very susceptible to disintegration by sea-water action. For reinforced work, as usually constructed, a slightly wetter mixture is required than what the writers believe to be the best consistency, owing to the necessity of thoroughly embedding the reinforcement.

It is possible, however, that some method may be devised of placing the concrete for reinforced work so that the ideal consistency can be used and, at the same time, secure thorough protection of the embedded steel. The ideal consistency is that which produces concrete of maximum density.

The author indicates that great care was exercised in proportioning the aggregates and in mixing the concrete, the time of mixing, after all the ingredients were placed in the drum, being about 2 min. This undoubtedly had a very beneficial effect on the concrete.

In many of the structures examined by the writers the deterioration in evidence, at least in part, can be attributed to a lack of care in preparing the concrete. More care must be taken in the mixing of the concrete than has too often been the case in the past, if more successful results are to be secured with concrete structures in sea water. In fact, far more care must be taken in every operation connected with constructing concrete which will be subjected to sea-water action than is necessary for land structures.

Messrs. Wig and Ferguson.

Only by giving strict attention to minute details, which in land structures would be trivial, can satisfactory results be obtained. Minor defects, which ordinarily would be unimportant, are often the starting points of deterioration which ultimately spreads to sound parts of the structures and cause failure when sea-water action is to be resisted.

Although the information which Mr. Bakenhus presents is of great value for the locality of Boston, care must be taken in drawing too general conclusions from it. There is a great difference in the action of sea water on concrete in different localities due to climatic conditions as well as varying physical actions, such as wave action and abrasion from floating débris and ice.

The saline content of the water also has a marked effect. The undiluted sea water along our coasts does not vary greatly in the quantity of salt contained, but, where structures are near the mouths of rivers, the sea water is frequently diluted to a considerable extent.

The writers believe that few very general statements can be made on the subject of the effect of sea-water action on concrete, but the peculiarities of each individual locality must be studied before attempting to state the methods to be followed, which will insure permanent concrete structures.

Mr. Bakenhus.

R. E. Bakenhus,* M. Am. Soc. C. E. (by letter).—The writer has read with interest the various discussions of his paper. Information is offered tending to confirm the results of the Boston tests. The opinion advanced by Mr. Yates that, in setting, a surface is formed on concrete, which will resist successfully the action of sea water on cement, is significant. If a surface coat protects the concrete from the chemical action by sea water, then it becomes very important to know the character of this protective surface, to encourage or bring about its formation, and to protect it from mechanical injury. Mr. Yates cites examples of bridge piers in fresh water showing no deterioration from freezing and thawing and no chemical action, though mechanical injury to the surface exists. This tends to bear out the theory that it is the chemical reaction between the sea water and the concrete that is responsible for most of the deterioration of the latter.

A number of causes are undoubtedly contributory to the deterioration of concrete in tidal sea water. It depends first on the destructive agents existing in the sea water and the atmosphere, and second on the materials of which the concrete is made and the manner in which they are combined. The active possible destructive forces may be listed as follows:

(*a*).—Mechanical injury by floating objects or débris;

(*b*).—Mechanical injury by water, waves, and currents;

(*c*).—Mechanical injury by wind action;

* Washington, D. C.

Mr. Bakenhus.

(*d*).—Mechanical injury from freezing and thawing;
(*e*).—Chemical reactions between the sea water and cement;
(*f*).—Chemical reactions between the sea water and the sand or stone of the concrete;
(*g*).—Physico-chemical reactions of the sea water and the concrete, such as the solution of elements of the concrete in sea water, or the formation and deposit, in the pores of the concrete, of crystals occupying greater volume than the original materials, accompanied by minute internal stresses;
(*h*).—A combination of any or all of these destructive effects.

The elements tending to resist destruction may be listed as follows:

(*a*).—Density of the concrete, that is, reduction of pores to a minimum, accompanied by increased resistance to internal stresses;
(*b*).—Thorough mixture of the ingredients, so as to give a uniform composition of the mass;
(*c*).—Freedom from accidental defects, which may act as starting points of deterioration;
(*d*).—Chemical and physical characteristics of the cement, sand, and stone, making these materials neutral in the tidal range of sea water;
(*e*).—Strengthening of the concrete by a high proportion of cement;
(*f*).—Formation of a protective coat, which is inert in the tidal range of sea water and which, unless mechanically broken, prevents chemical action.

Deterioration is the result of the interaction of the various destructive forces and the resisting elements. In the tests which have been described, the effort was made, by comparing parallel series of specimens, to single out and test the variable elements in the concrete itself, one at a time. The destructive elements of the sea and air might have been similarly segregated, and, in addition, the tests might have been repeated in various climates and in fresh water, as well as in salt water, so that the effect of mechanical injury from floating débris, the effect of wave action, the effect of freezing, the effect of climate and of moisture and air, and, finally, the effect of the chemical and physico-chemical action of the sea water might have been independently determined. In part, this is accomplished by the specimens extending below low tide and above high tide. In the absence of any more elaborate series of tests, the experiences with actual structures in many localities may supplement the data already available, and

Mr. Bakenhus.

enable engineers to reach definite conclusions. The writer, therefore, looks forward with interest to the report of the extensive investigations made by Messrs. R. J. Wig and Lewis R. Ferguson, for the Bureau of Standards, in all parts of the country.

In tidal sea water at Boston, the destructive effects, as shown by the tests, occur where the concrete is alternately exposed to the sea water and the air. Exposure to either the sea water or the air alone did not bring about deterioration. This would indicate that it is not simple chemical reactions between sea water and cement that bring about destruction of the concrete, as these could go on below tide level as well, but that the reactions are dependent on air as well as on water, and are most probably of a physico-chemical nature.

The effect of integral water-proofing on the durability of concrete in sea water is discussed by Messrs. Larsen and Rhett. The very interesting tests for the Grand Trunk Railway, at Portland, Me., referred to by Mr. Larsen, agree with the Boston tests in showing that concrete can be made without so-called integral water-proofing and remain in sea water for several years in perfect condition.

Two of Mr. Larsen's specimens, treated with external water-proofing, did not fare so well as the plain concrete; and six, made with cement, combined with various water-proofing compounds, are stated by him to show very marked deterioration.

Three of the Boston specimens were mixed with materials of the so-called integral water-proofing type. No. 22 had a mixture of 1 part of hydrated lime and 9 parts of cement. No. 23 was mixed with Sylvester wash, made as follows:

(*a*).—Bar soap, light colored, dissolved in water, 1¼ lb. of soap to 15 gal. of water;

(*b*).—Powdered alum, 3 lb., mixed with each bag of cement;

(*c*).—Materials of concrete mixed dry;

(*d*).—Hot soap solution dumped into a barrel of cold water, and used to mix the material to the proper consistency.

No. 24 contained pulverized clay in the proportions of 1 part of cement, $\frac{1}{20}$ part of pulverized clay, 3 parts of sand, and 6 parts of stone.

The clay specimen fared very well, but this is not true of the specimens containing hydrated lime and Sylvester wash.

The Boston tests are not sufficiently extensive as to the use of integral water-proofing to warrant drawing any general conclusions, and it cannot be stated, as a general principle, that they, in themselves, showed that integral water-proofing has a detrimental effect on concrete. Mr. Larsen's tests were more complete on this point, and show consistent results. The Boston tests do not include the

Mr. Bakenhus.

use of external water-proofing, such as was used in two of Mr. Larsen's tests.

Although most of the data available from these two series of tests are not favorable to the use of integral water-proofing, it may be possible, nevertheless, to discover a compound which would improve the durability of concrete in sea water. Very complete tests under service conditions will be necessary to determine finally whether a helpful integral water-proofing can be found.

It is shown by the tests that certain concretes are not stable in sea water at tide levels. Other concretes are stable in the presence of the forces, chemical and physical, that are active in sea water at tide levels. The dividing line between the two classes is not sharply marked. Knowledge of the action and effect of these forces is just as important to the engineer in designing successful concrete structures in sea water, as knowledge of earth pressure, hydrostatic pressure, or other loads. A very considerable volume of empirical knowledge is now available for the engineer from these tests and from other sources. Under existing conditions, it is apparent that concrete structures in sea water should be planned with care, and only by those who have special knowledge of the subject.

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Paper No. 1394

AN AERIAL TRAMWAY FOR THE SALINE VALLEY SALT COMPANY, INYO COUNTY, CALIFORNIA*

By F. C. Carstarphen, Assoc. M. Am. Soc. C. E.†

With Discussion by Messrs. Richard Lamb, H. F. Scholtz, and F. C. Carstarphen.

Synopsis.

This paper gives a brief statement of the prominent features of the location, construction, and operation of an aerial tramway built to carry salt, at the rate of 20 tons per hour, a distance of 13½ miles, from Saline Valley, over the Inyo Mountains, to a point in Owens Valley, California.

It is assumed that the reader is familiar with the terminology of aerial tramway design; also, that the ability of an aerial tramway system to transport materials economically and efficiently at the rate of 250 tons per hour or less is so well known as to make extensive discussion of these points unnecessary.

California now possesses one of the most novel aerial tramways ever constructed. This was built over the Inyo Mountains for the Saline Valley Salt Company, of Los Angeles, Cal.

In 1904 Adolf Bleichert and Company completed for the Argentine Republic, a tramway, 22 miles in length, with a difference in terminal

* Presented at the meeting of May 2d, 1917.
† Now M. Am. Soc. C. E.

elevations of 11 500 ft., thereby establishing a record for length of lines operated continuously and successfully. In 1915 the American Steel and Wire Company completed an aerial tramway for the Spring Canyon Coal Company, of Storrs, Utah, which carries 285 tons of coal per hour.

Although these two structures exceed the Saline Valley tramway in length and capacity, respectively, neither of them crosses a mountain range, nor does either show such boldness of engineering design. The engineers who undertook the construction of this tramway were fitted by experience to solve the many problems which confronted them. As early as 1895 the Trenton Iron Company designed a tramway, 12 miles long, for the Compagnie Haitienne, of Port de Paix, Haiti, and it has been maintained in successful operation ever since.

The purpose of the Saline Valley Salt Company, in building this aerial tramway, was to transport salt from Saline Valley, over the Inyo Mountains, to the railroad in Owens Valley. The tramway is in Inyo County which is remarkable as possessing the highest and lowest spots in the United States. Mt. Whitney, elevation 14 501 ft., is the crowning height of the Sierra Nevada Mountains, forming the west wall of Owens Valley; Death Valley, to the east, has an elevation of minus 280 ft., and is the lowest spot in the desert. An observer standing on the summit of Telescope Peak of the Panamint Range can view the extremes in elevation of the Republic.

It may be remarked in passing that Owens Valley, elevation approximately 3 600 ft., possesses a river of such magnitude that the City of Los Angeles has built an aqueduct, 274 miles long, for the purpose of making this mountain stream available for the domestic water supply of the city. The Owens River flows into Owens Lake, which is a dead sea, without outlet. The waters of this lake are sufficiently rich in sodium carbonate to make its extraction profitable. At present, plants on the east side of the lake are producing soda ash and sodium bicarbonate in large quantities.

To the east, beyond the Inyo Mountains, is Saline Valley, with an elevation of 1 100 ft. This valley has no water system, but is remarkable for several large springs, and there is flowing water in Hunter Canyon. To the east of Saline Valley lies the romantic Death Valley, its depressed floor dotted with borax deposits. Thus, it may be noted, the commercial worth of these valleys varies with altitude. The tempera-

ture of the summer air of Owens Valley is not excessive, but Saline and Death Valleys are no doubt quite the hottest places that can be found on earth. Temperatures of 120° Fahr. in the shade prevail for considerable periods, and only because of the absence of humidity can the summer heat be endured.

The topography of Inyo County owes its extreme boldness to the simple nature of the faulting, which is responsible for the Sierra Nevada and Inyo Mountains. When the great blocks of the earth's crust were thrust upward they assumed a tilt toward the west, so that the slopes on that side are gradual; the east fault plane has been carved into bold escarpments of exceeding grandeur. The elevations of the floors of the valleys between the successive mountain ranges indicate the relative power of the faulting forces. At the time of this great displacement of the earth's crust, titanic agencies were at work, and in regions of ancient volcanic activity deposits of precious ores are often found. This district is not an exception. In the early Eighties a gold stampede took place into the Saline Valley and the Ubehebe Mountains forming its eastern boundary. It is said that more than 2 000 people were camped in this valley during the height of the excitement. At that time it became generally known that there was a great salt deposit of remarkable purity in Saline Valley. A view of this deposit from the flanking mountains on the west is most impressive. The most important portion of the salt flat covers an area of approximately 1 500 acres, and the purity of the salt is enhanced by a natural refining process which is in constant operation, due to the remarkable fact that there is a series of large springs on the western rim of the salt flat, the water of which inundates the salt beds during the winter, and yields to the excessive evaporation induced by the summer heat. The retreat of the water deposits the pure salt crystals, which form when the brine reaches the point of saturation.

The salt claims located at that time have always been assumed to possess great value, provided transportation facilities could be obtained to ship the salt to the various markets.

Surveys had demonstrated the impracticability of building a railroad into the valley; the mountain barrier was so great that the cost of constructing such a line was excessive. Two methods of transportation were then considered. The first was by a pipe line over the Inyo Range, through which the brine formed on the salt flat could be

pumped to evaporating vats adjoining the railroad at Owens Valley. This project was examined very carefully, and estimates of cost were prepared, but it was found that although it was feasible for the transportation of brine, it did not afford a means of bringing in the supplies needed by the operations incidental to pumping. Accordingly, the merits of an aerial tramway were discussed, and, in order to ascertain the cost, and determine a suitable location for such a tramway, a survey was begun in April, 1911, under the direction of W. H. Leffingwell, M. Am. Soc. C. E. Several trial lines were run, and a location was selected in the latter part of May. Under the direction of Mr. C. H. Wickham, Field Engineer for the Trenton Iron Company, a final location was completed in July. The profile, together with the topographical surveys, were submitted to the Trenton Iron Company for consideration. An estimate was prepared, and the final contract was signed on August 14th, 1911, between The Trenton Iron Company, a subsidiary of the American Steel and Wire Company, and the Saline Valley Salt Company, a corporation of the State of Arizona.

This contract called for the preparation of the designs and finished drawings, as well as the wire cables, carriers, and machinery, for a Trenton-Bleichert tramway to carry salt weighing 60 lb. per cu. ft., the slope length of the line being 69 645 ft., the capacity 20 tons per hour, the elevation of the discharge terminal above the loading terminal, 2 450 ft., the carriers to have a volume of 12 cu. ft., and the speed of the traction rope to be 500 ft. per min. This resulted in a spacing of 525 ft., or 63 seconds, between contiguous carriers. More than 112 h. p. were required for its operation. The following items occur as prominent parts of the specifications:

Patent locked coil steel track cable, $1\frac{1}{4}$-in.	13 850 ft.
" " " " " " $1\frac{1}{8}$-in.	55 450 "
" " " " " " $\frac{7}{8}$-in.	69 300 "
Special steel traction rope, $\frac{3}{4}$-in.	141 000 "
Special steel buckets, capacity 12 cu. ft., with protecting covers; and hangers, carriages, and patent compression grips	286
Intermediate supports	120
Track cable saddles	240
Steel traction rope rollers, with shafts, bearings, bolts, washers, foot castings, etc.	240

The specifications also included the supervision of the erection of the timber work and attaching the machinery; the equipment for one loading terminal, one discharge terminal, five intermediate control stations, twenty-one rail structures, twelve anchorage-tension structures, and one double-tension structure; tools for coupling and splicing track cables and traction rope; the necessary signal gongs for the control of the line when in operation; track cable oiling cars, traction rope oiling tanks, and water carriers.

The difficulties of construction on this line may be inferred from an inspection of the profile, Fig. 1. Aerial tramways have been built in the past so that the line traversed country possessing exceedingly rough topography, but each has been distinguished by the fact that it has been on the flank of a mountain range, instead of crossing it. It was desirable to find the shortest route between the salt fields and the railroad. The west side of the Inyo Mountains, though broken into a number of precipitous ridges, does not impose extreme difficulties to aerial tramway erection. The average elevation of the crest of these mountains is 10 000 ft. above sea level, and it runs in a northerly and southerly direction. However, there is a saddle in the crest line with an elevation of 8 500 ft., so that it was desirable to have the tramway pass through this gap. By utilizing the difference in elevation between the terminal stations, it was found that the 7 500 ft. required to pass the summit could be obtained by dividing the difference in altitude between the floor of Saline Valley and the summit into three 2 500-ft. sections; also, that the descent from the summit to Owens Lake could be made in two 2 500-ft. sections. The selection of a constant difference in the height of the several control stations was an important factor in deciding on a constant size of traction rope for all the sections. The use of intermediate control and driving stations added flexibility to the location of the line, as any reasonable horizontal angle can be made at such points.

The final location of the line is shown on the map, Fig. 2. It starts from the edge of the salt field, at an elevation of 1 060 ft. and runs south along the mesa for 2¾ miles to the foot of the eastern face of the mountains, which has an elevation of about 1 800 ft. The line ascends abruptly to Station 177 + 50, which has an elevation of 3 720 ft., where Control Station No. 1 is placed. The portion of the line from the loading terminal to Control Station No. 1 is designated

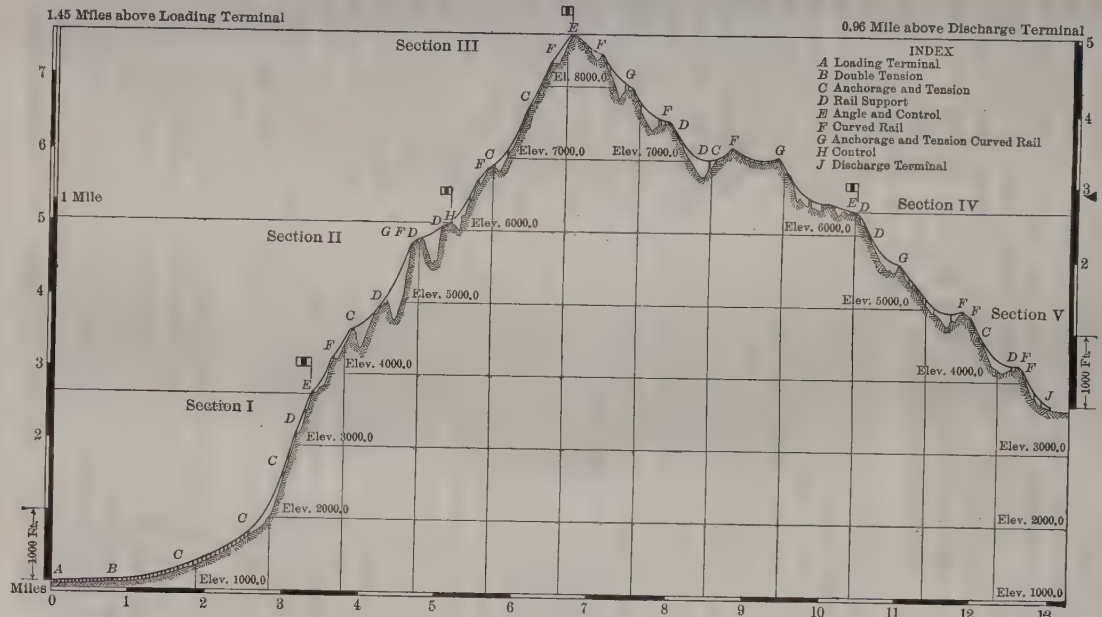

FIG. 1.—PROFILE, AERIAL TRAMWAY, SALINE VALLEY SALT COMPANY, TRAMWAY, CAL.

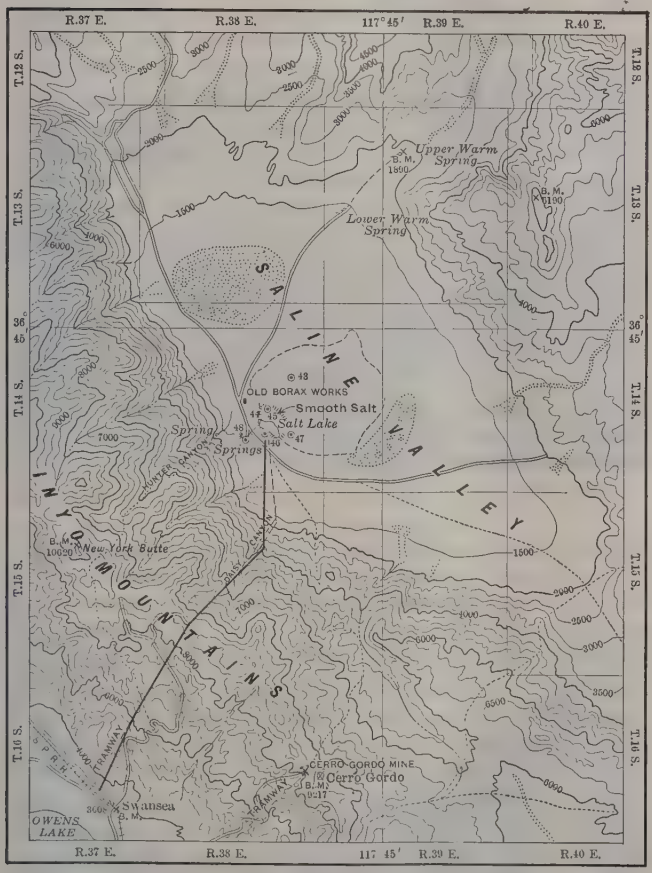

FIG. 2.—LOCATION OF AERIAL TRAMWAY OF SALINE VALLEY SALT COMPANY (U. S. G. S. TOPOGRAPHY).

Section I. A horizontal angle of 35° 30′ is turned to the right, or westward, at this control station. The line crosses Daisy Canyon between Stations 223 and 244, and attains an elevation of 6 100 ft. at Station 270, where Control Station No. 2 is established. This is known as Section II. The line continues without angle to the summit of the mountain. Control Station No. 3 is at this point, 8 300 ft. from Control Station No. 2. This is Section III. There is an angle of 2° 28′ to the left at this point. Control Station No. 4 is at Station 550 + 66, elevation 6 330 ft., and is the limit of Section IV. A horizontal angle of 10° 48′ is turned to the left to Station 685 + 63, at an elevation of 3 625 ft., the site designated for the discharge terminal, which adjoins a spur of the narrow-gauge branch of the Southern Pacific Railroad, terminating at Keeler, Cal.

The rugged nature of the topography is shown by Figs. 3 to 14, as well as by the plat of the survey. At the time the survey was made there was great difficulty in reaching the points traversed by the line. In numerous cases, when the survey parties were working in deep canyons, days were consumed in finding a feasible path of ascent to the summits of the cliffs.

Construction was started on September 1st, 1911, when the framing of the timber for the towers on Section IV was commenced. While the framing was in progress, new roads and trails were being built, so as to command the location of the line. At a point on the railroad about 3½ miles north of Keeler an old charcoal road traverses the west slope of Inyo Mountain to within a mile of the summit crossing. The summit was connected to the old road by a new construction, and the old road was repaired and made serviceable. However, at an elevation of 5 900 ft., there was a stretch of about 330 ft. of road having a grade of 25%, and the conditions were such that this grade could not be eliminated or improved at a reasonable cost. Thus, 25% became the ruling grade for hauling the material needed for the construction between Control Stations 4 and 1. The maximum load which eight horses can haul on such a grade is about 5 000 lb. Heavier loads required ten or twelve horses. As the distance from Owens Valley to the summit was nearly 10 miles, a camp was established on the road about 3½ miles west of the summit. By this arrangement teams could make one trip per day from the railroad to the camp, leaving their loaded wagons for other teams, quartered at the camp,

FIG. 3.—SALINE VALLEY AND STRUCTURE 9. ANCHORAGE AND TENSION STRUCTURE 11.

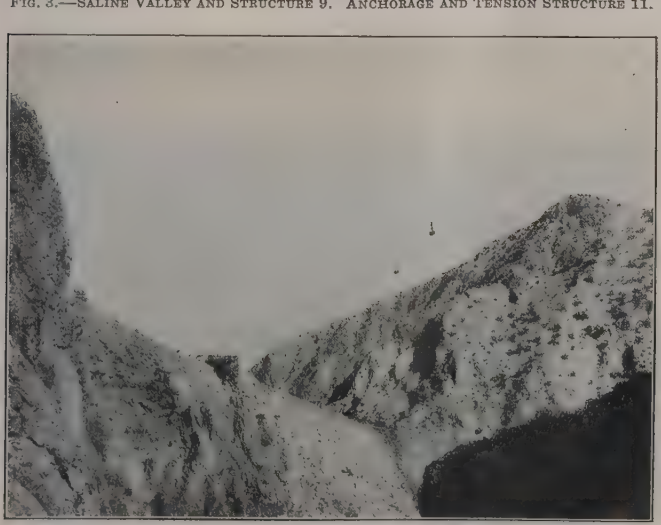

FIG. 4.—TRAMWAY CROSSING DAISY CANYON, CONTROL STATION 3 TO STRUCTURE 16. LINE RIDER ON CARRIER.

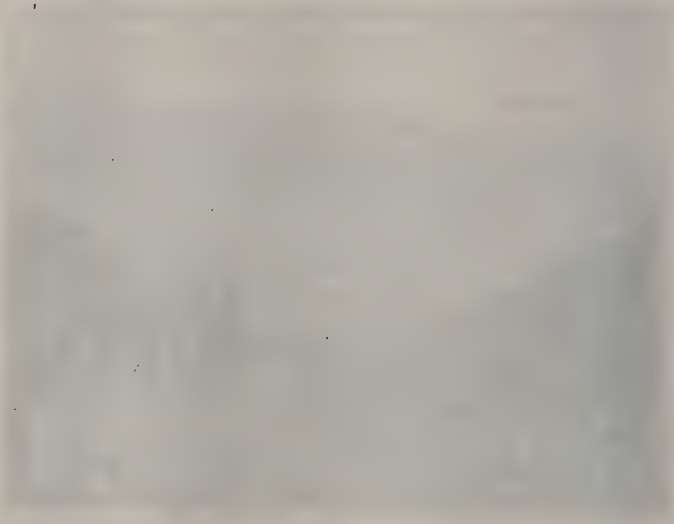

FIG. 5.—BUCKET LEAVING LOADING TERMINAL.

FIG. 6.—LOADED AND EMPTY CARRIERS PASSING ON 80% GRADE, SHOWING VERTICAL POSITION MAINTAINED BY BOTH.

to haul to the summit and unload. The camp teams could make three trips in 2 days. The total weight of material hauled from the railroad to this camp was found to be 1.1 tons of grain, 5.4 tons of hay, and 23.2 tons of water, for each 70.3 tons of material required for the tramway construction. The maintenance cost of a team, consisting of eight horses and driver, with headquarters at the railroad, was $15 per day. A team maintained at the upper camp cost $21 per day, the increase being due to the fact that all feed and water used at the camp had to be hauled from the railroad.

It was impossible to build roads on the east flank of the Inyo Mountains. After a great deal of difficulty, a trail was constructed over the summit to Saline Valley. As heavy machinery could not be moved by pack animals, it was necessary to construct temporary, double-cable, reversible tramways for this purpose. The diameter of the track cables used was ¾ in. and that of the traction rope, ⅝ in. Suitable carriers were improvised by using two or more standard tramway timber carriers. The weight of the material transported by these jigbacks was approximately as follows: From Control Station No. 3 at the summit, for Structures Nos. 19, 18, 17, 16, and the towers in Section III, about 60 tons, at a cost of from $2 to $4 per ton, the difference being accounted for by the fact that the material was unloaded from the line as required: From Structures Nos. 20 to 15, a distance of 8 300 ft., 600 tons were moved, at a cost of $4.20 per ton; from Structures 15 to 11, 2 600 ft. distance, 375 tons at $2 per ton; Structures 11 to 9, 4 000 ft., 260 tons at $3.50 per ton; Structures 9 to 7, 2 700 ft., 210 tons at $2.90 per ton.

The cost per day for the crew required to operate, load, and unload these temporary cableways was from $35 to $42. The capacity of these lines, in tons per day, when operating, may be summarized as follows: From Structures 20 to 15, 13.3 tons; 15 to 11, 33.3 tons; 11 to 9, 23.8 tons; 9 to 7, 29.7 tons. If the capacity is stated in tons per day, including the time of putting up, taking down, and moving the jigbacks, the figures are as follows: Structures 20 to 15, 6.3 tons; 15 to 11, 19.3 tons; 11 to 9, 11.3 tons; 9 to 7, 14 tons.

From Chuckwalla Hollow, between Structures 4 and 5, a line was built to Control Station No. 1. It cost $35 a day for the crew, and took 6 days to build. One trip per day could be handled with this line,

at an average cost of $50. If the material was light, such as wood, water, etc., two trips could be made. This meant a labor cost of more than $25 per ton for handling motors and other heavy pieces of machinery from Structures 4 to 7. This expense is accounted for when it is recalled that the vertical lift is 1 900 ft. in a horizontal distance of 3 500 ft.

Saline Valley can be reached by a wagon road, approximately 55 miles long, which leaves the railroad at Big Pine. About 375 tons were hauled over this road at a cost of $35 per ton. All the cables required between Stations 1 and 2, together with the 75-h.p. motor, transformers, and heavy station material, were moved from the summit to a point in Daisy Canyon about 1 500 ft. west of Control Station No. 2. This material was transported on what is called a "go devil." This device consisted of a timber frame supported on an axle 8 ft. long which passed through two heavy wheels. The rear end of the frame rested on the ground, and was shod with iron in order to prevent excessive wear. On each side of the frame, near the rear end, steel hooks were arranged, so that two men, one on each side of the frame, could lift it free from the road when the gradient was light. On the other hand, these hooks were used to check the velocity of the device when traveling over ground that was soft enough for them to penetrate. The operators used both hooks in coming down steep places, and also assisted the team in steering the device. Fortunately, no road was required for the operation of this machine, as the canyon was fairly straight and the bottom free from ledge rock.

The material was moved to Control Station No. 2 on a tramway constructed with two carrying cables mounting four carriers. A different system was adopted for stringing the cables from the summit to Control Station No. 2, and also from No. 2 to No. 1. This consisted of coupling cables together and letting them down the mountain side by gravity. It was necessary to keep from three to five men at the leading end so as to put the cables on the traction rope guide sheaves of the towers and stations. The velocity of the cable was controlled with 4 by 8-in. blocks bolted together, so that the cable could be squeezed as it passed along. Each one of these extemporaneous brakes would resist a pull of about 1 000 lb. until a groove equal to the depth of the cable was worn in it. When this occurred the clamp was changed so

FIG. 7.—THE 2 400-FT. SPAN. CONTROL STATION NO. 1 IS AT THE TOP, UNDER THE CROSS.

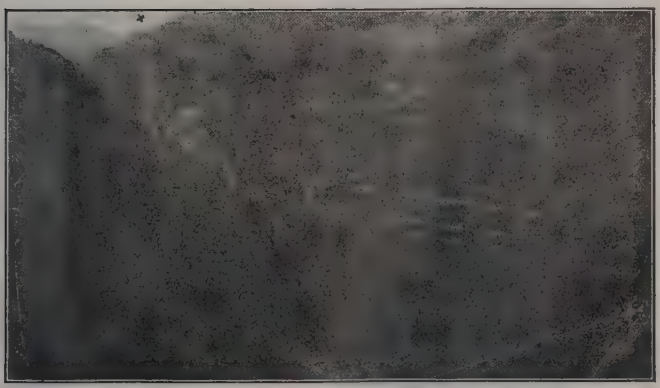

FIG. 8.—TRAMWAY CROSSING DAISY CANYON, STRUCTURES 10 TO 11 ; 2 260-FT. SPAN ; 600 FEET ABOVE BOTTOM ; 85% GRADE INTO STRUCTURE 11.

FIG. 9.—THE SALT FIELD.

FIG. 10.—CONTROL STATION NO. 1, AND DISTANT VIEW OF SALT FIELD. NOTE THE REVERSE VERTICAL AND HORIZONTAL CURVES IN THE STRUCTURES.

that the cable would bear on a new spot. From Structure 4 to Control Station No. 1 the cables were pulled up with a winch driven by a gasoline engine. On Sections IV and V the cables were strung and placed under tension in the usual manner, except that on Section V the grade was sufficient to make it desirable to use the gravity method. In placing the traction rope for Sections III and II, it was threaded around the grip sheave at the summit station. The grip sheave acted as a driving sheave, due to the use of a brake on the reel, which imposed tension in the traction rope before it passed around the grip sheave. The traction rope was also passed through hauling clamps before reaching the grip sheave so as to hold it when splices were being made, or in cases of emergency. After getting the traction rope for one side to Control Station No. 1, the upper end at Station 20 was snubbed, and the rope for the opposite side was let down in the same manner. After the upper ends had been spliced, the ropes were cut at Control Station No. 2 and passed around the tension and grip sheaves of this station.

The towers and structures are carried on concrete foundations, and the water required for making the concrete was hauled in tanks or packed by mules for distances ranging from 1 to 7 miles. This was also true for the water required for the camps, except those at the railroad, in Daisy Canyon, and Saline Valley. All permanent camps on the east slope were at elevations lower than 6 000 ft., and those on the west slope lower than 7 500 ft., so as to be below the usual snow line. As the mountains are extremely barren, firewood had to be hauled or packed to the camps from the most convenient points, and these were often at considerable distances.

From October 1st to April snow storms are of frequent occurrence, and, owing to the steepness of the hillsides, constitute a menace to construction at all altitudes above 6 500 ft. In the spring occasional slides have occurred on the eastern slope, and have proved disastrous to the construction crews. The drifting snow was very objectionable, owing to the difficulty of keeping the trails open. Due to these several causes, the work was arranged so that it could be performed at elevations below the snow line during the winter.

The erection of the line was pushed energetically, and the tramway was ready for the preliminary test on June 10th, but the electrical con-

nections from the power-house of the Los Angeles Aqueduct, which is in Cottonwood Canyon on the opposite side of Owens Lake, were not completed until later. The first bucket of salt arrived at the discharge terminal on July 2d, 1913, and was the occasion for a great demonstration.

The placing of salt at the railroad was a great achievement, and taxed to the utmost the financial capacity of the Saline Valley Salt Company. The reasons for this stringency may be indicated as follows: After the contract was signed, the question of the location of the discharge terminal was an open one for the next 7 months. This indecision as to the terminus of the tramway affected the schedule which was prepared to guide the manufacturer in fabricating and shipping essential parts. It also deranged the plans of the construction crew. Nothing could be done in the making of surveys and the preparation of profiles, designs, or drawings until the site of the discharge terminal was known. Because of this confusion the completion of the contract was delayed beyond the intended date, so that the Salt Company was required to meet the expenses of a construction organization for a longer period than was contemplated in the original estimates.

The contract specified that the capabilities of the tramway should be demonstrated by an operating test of not more than 60 days, under the supervision of the engineers of the buyer and seller. To carry out this provision, active steps were taken by the field engineers to place the specified number of carriers on the line and to co-ordinate the various functions of the machinery, carriers, and signal systems so that the operating test could be undertaken as speedily as possible.

The line worked with great satisfaction when the buckets were loaded about two-thirds full. When efforts were made to carry full buckets, it was noted that the number of accidents due to runaway carriers and others slipping on the traction rope was excessive. An examination of the equipment was immediately started to ascertain the cause. After carefully canvassing all the agencies which might have influenced this unusual behavior of the carriers, the question of the weight of the salt as furnished to the tramway was raised, and an investigation was undertaken to determine the facts. The contract called for the transportation of salt weighing 60 lb. per cu. ft. The

FIG. 11.—TRAMWAY CROSSING DAISY CANYON. ALL PACK TRAINS MUST PASS THROUGH THIS NARROW DEFILE.

FIG. 12.—VIEW OF SALT FIELD FROM CONTROL STATION NO. 1, DIFFERENCE IN ELEVATION OF FIELD AND STATION = 2 600 FEET.

FIG. 13.—BUCKET OF SALT AT LOADING TERMINAL.

FIG. 14.—FLEXIBLE HOODS CONNECTING TRACK CABLE AND DOUBLE-HEADED TRAMWAY RAIL.

actual weights of the salt, as determined by careful tests at the salt field, were as follows:

Air-dried salt	60	lb. per cu. ft.
Damp salt	65	" " "
Salt sufficiently moist to agglomerate when squeezed in the hand	68.5	" " "
Salt when brine is dripping slowly	83.3	" " "
Salt when brine is dripping freely	90.2	" " "
When brine is running from salt	94.2	" " "

It was also noted that the average run of salt from the field was of the wet variety, so that its weight was in excess of 80 lb. per cu. ft. As the weight of saturated salt exceeds by more than 50% the figure stated in the contract, it is evident that the stresses imposed on the tramway had a similar range in value. Unfortunately, they were increments to the stresses contemplated in the original plan. The use of liberal factors of safety in design is again justified, for otherwise this line would have failed under the increased burden imposed. When these conditions were discovered, careful observation of the behavior of the carrier showed that, when carrying dry salt, they traversed the line from the loading to the discharge terminals without noticeable difficulty, but when loaded with wet salt, a few of the buckets would slip on the traction rope. The points where this difficulty was particularly pronounced were between Structures 5 and 6 of Section I; and between Structures 10 and 11 of Section II. There the track cables are as steep as, or steeper than, can be found on any operating aerial tramway using friction grips. In order to indicate the excessive gradients which confronted the designers of this tramway, Table 1 is presented. All gradients of less than 30% have been omitted. This table shows the station numbers of the approved survey, the numbers of the structures, the slope and length of the chord of the span, the number of loaded carriers supported on the span, and the slope, in angular measure and in percentage. These data will be found of great interest, especially to engineers concerned in aerial tramway design. Those who study Table 1 will realize the difficult problems these grades imposed on the designers of the tramway.

To know that this aerial structure is in successful operation is a source of deep gratification to its sponsors. The first great departure

TABLE 1.—Elements of the Aerial Tramway of the Saline Valley Salt Company.

Span. From	Span. To	Slope of chord.	Length of chord, in feet.	No buckets in span.	Slope of cable.	Percentage of slope.
Saline to Summit.						
135 + 50 (4)	158 + 50 (5)	20°06′	2 450	5	39°20′	81.95
158 + 90 (4)	162 + 48	30°53′	.....	1	36°17′	73.41
162 + 48	164 + 38	31°40′	.....	1	36°28′	73.9
164 + 38	166 + 68 (6)	32°16′	Entering No. 6	1	39°45′	83.17
168 + 89 (6)	173.	30°27′	709	2	37°49′	77.61
173.	176 + 03 (7)	31°51′	Entering No. 7	1	39°36′	82.73
178 + 57.3 (7)	191 + 07.7 (8)	20°18′	1 335	3	34°39′	68.73
191 + 34 (8)	203 + 48 (9)	18°19′	1 280	3	33°15′	65.5
219 T.	223 + 08 (10)	15°42′	500	1	20°37′	37.62
223 + 28 (10)	244 + 10 (11)	22°32′	2 260	5	40°27′	85.25
244 + 63 (11)	247 + 22 (12)	11°32′	.....	1	19°08′	34.69
251 + 31 (13)	265 + 91 (14)	6°18′	1 459	2	21°24′	39.19
271 + 10 (15)	284 + 50 T.	15°44′	1 392	2	28°11′	53.58
284 + 50 T.	289 + 11 (16)	23°07′	500	1	31°00′	60.0
289 + 41 (16)	295 + 81 (17)	17°26′	513.5	1	25°23′	47.45
301 + 30 T.	310 + 35 T.	13°30′	931	2	22°36′	41.62
310 + 35 T.	318 + 50 T.	20°58′	873	2	29°36′	56.8
318 + 50 T.	320 + 70 T.	24°25′	.....	1	29°20′	56.19
320 + 70 T.	322 + 94 (18)	24°32′	.....	1	31°35′	61.5
323 + 33 (18)	325 + 30 T.	20°06′	.....	1	24°57′	46.52
325 + 30 T.	330 + 00 T.	21°18′	508	1	27°06′	51.17
330 + 00 T.	336 + 00 T.	23°54′	656	2	30°58′	60.0
336 + 00 T.	338 + 29 (19)	26°06′	.....	1	33°30′	66.19
339 + 02	351 + 95	16°42′	1 352	..	31°13′	60.6
Summit to Discharge Terminal.						
374 + 46 (21)	377 + 20 T.	18°31′	.....	1	26°10′	49.1
377 + 20 T.	379 + 00 T.	18°33′	.....	1	23°20′	43.1
379 T.	390 + 00 T.	12°31′	1 127	3	23°01′	42.5
396 + 15 (22)	399 + 90 T.	19°11′	.....	1	26°44′	50.3
399 + 90 T.	400 + 90 T.	17°14′	.....	1	21°45′	39.9
400 + 90 T.	413 + 50 T.	10°46′	1 285	3	22°37′	41.6
421 + 51 (23)	423 + 70 T.	20°49′	.....	1	25°44′	48.2
423 + 70 T.	426 + 91 (24)	19°48′	.....	1	25°05′	46.8
427 + 13 (24)	448 + 20 (25)	8°10′	2 128	1	26°41′	50.3
497 + 40 (28)	503 T.	19°30′	594	2	28°23′	54.0
503 T.	504 + 50 T.	17°45′	.....	1	22°26′	41.3
504 + 50 T.	506 + 30 T.	17°54′	.....	1	22°41′	41.8
506 + 30 T.	517 + 40 T.	12°06′	1 135	3	22°45′	41.9
551 + 75 (29)	553 + 10 (30)	16°29′	.....	1	23°14′	42.9
553 + 26 (30)	556 + 50 T.	19°27′	.....	1	27°51′	52.8
556 + 50 T.	557 + 50 T.	19°08′	.....	1	23°39′	43.8
557 + 50 T.	558 + 50 T.	19°57′	.....	1	24°28′	45.5
558 + 50 T.	561 + 34 (31)	20°12′	.....	1	25°21′	47.4
561 + 57 (31)	580 + 68 (32)	10°33′	1 945	4	29°26′	56.4
581 + 46 (32)	584 T.	15°35′	.....	1	22°45′	41.9
584 T.	588 + 10 T.	14°18′	.....	1	19°53′	36.2
588 + 10 T.	590 T.	18°19′	.....	1	18°08′	32.75
590 T.	591 + 28 T.	13°21′	.....	1	17°58′	32.4
591 + 28 T.	592 + 50 T.	13°35′	.....	1	18°10′	32.8
592 + 50 T.	593 + 77 T.	14°02′	.....	1	18°38′	33.7
593 + 77 T.	595 T.	14°37′	.....	1	19°12′	34.8
595 T.	597 + 50 T.	14°50′	.....	1	19°42′	35.8
597 + 50 T.	600 T.	14°37′	.....	1	18°39′	33.8
600 T.	605 + 80	11°44′	.....	1	18°19′	33.1
631 + 88 (34)	633 + 20 T.	22°32′	.....	1	28°17′	53.8
633 + 20 T.	687 + 30 T.	21°34′	.....	1	27°09′	51.3
687 + 30 T.	638 + 95 (35)	20°53′	.....	1	25°37′	47.95
639 + 30 (35)	659 + 26 (36)	9°27′	.....	4	28°23′	54.0
663 + 97 (37)	667 + 21 (38)	13°11′	.....	1	20°35′	37.55
667 + 76 (38)	675 + 40 T.	19°05′	.....	2	27°58′	53.1
675 + 40 T.	680 + 90 T.	18°08′	.....	2	19°30′	35.4

from standard tramway practice was made when the extreme difference in elevation between the loading terminal station and the summit was divided into three sections, and when two sections were used for the descent from the summit to the discharge station. Experience has shown that it is not economical to attempt to utilize traction ropes of large diameter. The total stress developed in such ropes when lifting 20 tons of material 1½ miles vertically at the rate of 500 ft. per min. is so great as to prohibit the use of a single rope for this duty. Multiple traction ropes, arranged in parallel, are not to be considered when a more satisfactory solution of the problem can be developed. Accordingly, it was decided that the tramway should be arranged in sections so as to maintain a traction rope of moderate diameter, and to arrange

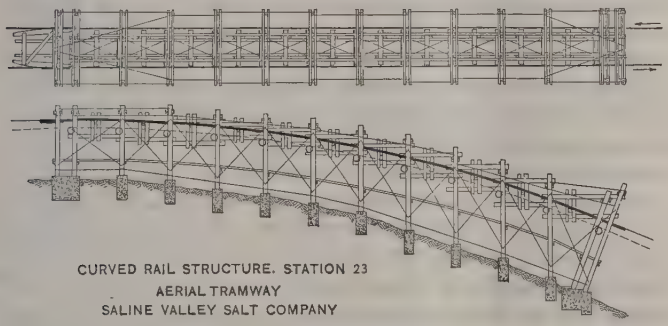

FIG. 15.

the stations so that there would be a constant difference in their successive elevations. It is readily seen that this constant difference in elevation imposes practically the same stresses on the traction ropes of each section. The only feature that is not constant is the friction developed by the moving carriages on those sections which are not of equal length and, consequently, do not support the same number of carriers.

Unfortunately, experience has shown that, irrespective of the constancy of tension in two traction ropes in service, they do not maintain precisely the same diameter. Accordingly, friction grips which have been designed so as to maintain a uniform closure when adjusted to hold satisfactorily on the larger of the traction ropes will slip, all other conditions being constant, when attached to the second traction rope.

By this criterion it was necessary to abandon consideration of the well-known Webber grip for this service. The use of either the Bleichert overhead or underhung automatic grips was taken under advisement. The screw grip, such as Pohlig uses, was unsatisfactory because of the acceleration required to close the gripping lever, which moves in a plane parallel with the direction of motion; also because of the great difficulty in maintaining a proper limit for the throw of the closing lever when operating on ropes of different diameters so as to engage the detacher properly instead of under-running it, an accident which happens frequently on multiple-section lines where this grip is used. The overhead grip was eliminated from further consideration because the extreme tension of the traction rope in passing through structures on the crests promoted the overturning tendency of the carriers, especially those on the empty side. The underhung Bleichert grip was not used on account of the serious doubts entertained concerning its holding powers on grades as steep as 86 per cent. This grip is operated by the weight of the load acting on the movable jaw. On steep gradients the gripping power is materially reduced. The high price of the grip and carriage of this type, as compared with others, was also a considerable factor when it is recalled that 286 carriers were needed for the equipment of the line. As all the accredited grips then known were found to be unfit for this service, it was decided that a new grip should be designed so as to hold carriers supporting 12-cu. ft. buckets loaded with salt weighing 60 lb. per cu. ft. It was also decided that these grips should be of the trunnion type, so that the loads would hang vertically on all slopes; that the grips would compensate successfully without material loss of tractive effort for the different diameters of traction ropes found on the line; that the grips should be of the underhung, top-opening type; and that they should be of moderate cost.

When these grips were furnished and the line was operated, it was found that they fulfilled most of the aims of the design, but, owing to the great variation in the weight of the salt and the consequent increase in drag on the grip, some of them failed by slipping on the extreme slopes. In other words, the requirement for the constant adjustment of grips was too great a hazard to success to comply with the severe specifications imposed by the engineers of the American Steel and Wire Company, who ruled that, because of their uncertain performance when handling the heaviest loads possible, they must be replaced. This

conclusion was reached only after a considerable lapse of time from the date of arrival of the first bucket of salt at the discharge terminal, the reason being that the Salt Company experienced grave financial difficulties which, for about 2 years, prevented running the tramway in accordance with the contract.

Finally, the American Steel and Wire Company voluntarily took over the operation of the line at its own expense, so as to determine what defects, if any, existed, and what replacements should be made. As a result of this investigation, a new grip, called the Universal Wico, was perfected, which has demonstrated its fitness for the duty required. It is designed in strict accordance with the specifications previously outlined, and has sufficient gripping power on the traction rope to handle safely on the steepest grades a carrier loaded to its utmost capacity with the heaviest salt plus the weight of a line rider. It has the additional property of requiring a very light force on the closing lever, and is free from all jars so common to other forms of grips, due to the elasticity imparted to its action by the floating pivot of the closing lever. The grip requires about 40 lb. on this lever to close it. As the carrier has to be attached and detached five times during the passage from the loading to the discharge station, the troubles incident to these operations are five times as great as those on single-section lines, the usual form of the great majority of aerial tramways. In making a round trip on this line the carriers travel more than 26 miles, and a special lubricating system is attached to them so as to provide an adequate supply of oil for the wheels and pins.

Section I is exceptional. From the loading terminal to Structure 4, a distance of 13 550 ft., the line crosses a gradually rising mesa which serves as a fillet between the flat bottom of the valley and the abrupt escarpments of the fault cliffs of the mountain. From Structure 4 onward, the line rises 800 ft. with great abruptness. It is evident that in hauling carriers up a line of this contour the track cables and traction rope must be arranged so that the latter will never carry sufficient tension to cause it to rise above the track cables. Such a condition would be disastrous, as the carriers would be derailed. After careful computations, the tensions were determined so that this section operates perfectly.

Control Station No. 1 is unique in that it is equipped with driving mechanism, tension sheaves, guide sheaves, and a horizontal angle;

and the ends of the building are developed into a pronounced rail structure.

Control Station No. 2 is also equipped with driving mechanism, and presents a reverse vertical curve in profile.

The summit is unusual in that Sections III and IV are controlled from this point, both driving sheaves being mounted on the same shaft. (Fig. 16.) Therefore, the power developed by the descending loads on Section IV assists in raising those on Section III.

As the carriers are being lifted on Sections I, II, and III, there is a possibility of the line reversing its direction of motion, due to the unbalanced weight of the carriers on the loaded and empty sides. Accordingly, each of these stations is equipped with a reverse preventer. This device consists of a large circular plate fitted with shoes containing ball races under the control of a centrifugal governor. When the governor is up to speed, the balls are restrained so that the plates run freely. When the line stops, the balls immediately nip the plate, and a reversal of motion is prevented.

Control Station No. 4 is also equipped with a reverse preventer to stope a retrograde motion of the line in case it is loaded with "back" freight destined for Saline Valley.

The drives of these stations are similar, except that at the summit two sheaves are mounted on the same shaft, as before mentioned. They consist of an 8-ft. grip sheave bolted to a brake run, 7 ft. 2 in. in diameter, with a 12-in. face, and this, in turn, carries an 8-ft. spur gear. The latter is driven by an 18-in. pinion mounted on a shaft fitted with a friction clutch belted to a 75-h.p., Allis-Chalmers, 3-phase, 2 200-volt, induction motor. On Section V this motor acts as a dynamic brake, instead of a driver. A grip sheave is one which carries a special rim, in which there are about ninety-six pairs of toggle jaws. These jaws are held open by gravity or by a special flat spring. When the traction rope passes around the sheave, the pressure developed on the bottom of the groove of these jaws is sufficient to cause them to close and grip the rope tightly. This action increases the friction between the sheave and the traction rope, thereby greatly promoting its efficiency.

The brakes are of the compound differential type, and are actuated by hand-wheels and screws; this method ensures that they will be applied gradually during an appreciable interval of time, so that the

line will not be wrecked because of sudden stoppage, as would be likely to occur with solenoid brakes.

The carriers are of special design, in that the hangers are made longer than is standard, so as to provide proper clearance between the traction rope guide rollers used in the rail structures on the ridges running transversely to the location of the tramway.

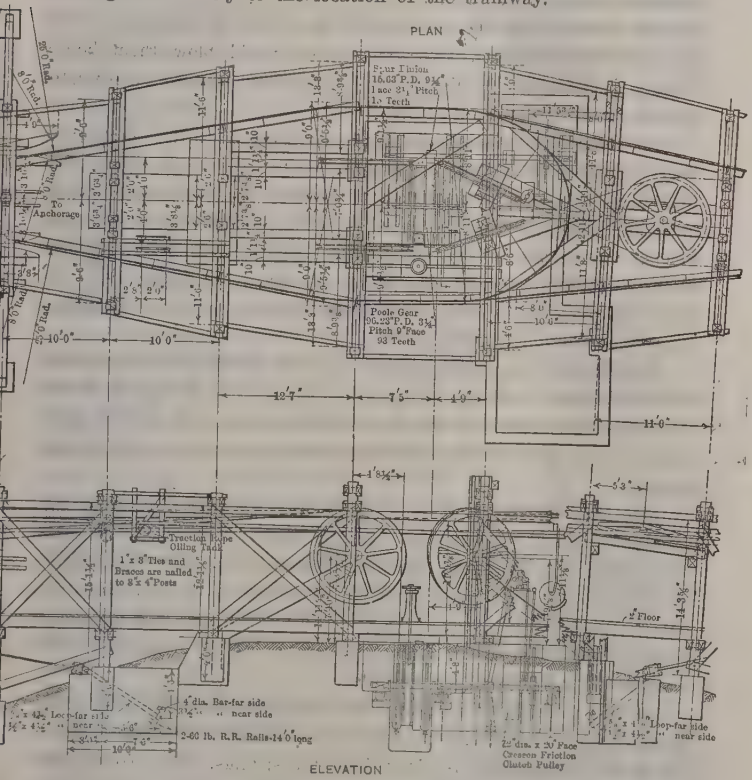

FIG. 16.—PART OF CONTROL STATION NO. 3, SHOWING DRIVES FOR SECTIONS III AND IV.

Owing to the extreme length of the line, it was necessary to cover the salt in order to protect it from contamination due to dust and grease which might drop from the track cable. To obviate the necessity of removing this cover whenever the bucket is dumped, the latter

is cylindrical, and revolves about its trunnions independently of the cover. Owing to the vibration of the carrier in passing through such a great number of stations and structures, it was necessary to provide a very positive keeper to hold the bucket latch in place. In the locality contiguous to the tramway site there are winds of great violence, promoted, no doubt, by the high temperature in the valley and the cooled air of the summit. If the covers of the buckets were not properly restrained, these hurricanes would blow them bodily through the hangers; therefore, it was imperative to design a dependable latch to hold the covers in place.

The control stations are equipped with telephones, thus permitting ready communication from one end of the tramway to the other. The line riders, who are responsible for the oiling of the traction rope rollers and the track cables of the several sections, are provided with line 'phones which can be attached to the wires conveniently, thus permitting conversation with any of the stations from points along the line. There is also a bell signal system for use in those operations where conversation is not required.

The electrical transmission line is of the usual pole construction, carrying copper wires for a 3-phase circuit. The voltage is 33 000. The transformer stations are of the out-door type, and the voltage is stepped down from 33 000 to 2 200 for use in the motors of the driving stations.

The crew consists of two men each at the loading and discharge terminals, as well as at Control Stations Nos. 1, 2, 3, and 4. In addition to this crew, four line riders are required to keep the track cables and the traction rope rollers and guide sheaves in the structures and towers in perfect condition. There is also a foreman who exercies a general supervision of the line. The cost of labor and power for salt transported is 4.6 cents per ton-mile.

Several methods of gathering the salt have been tried with indifferent success. At present the plan is to flood the salt field by manipulating the water which is piped from the springs. This fresh water dissolves the salt rapidly. If the supply of fresh water is then stopped, the evaporation caused by the summer sun is excessive, and pure salt crystals gather in the brine. These crystals resemble flat plates, and are easily shoveled into conical piles about 2 ft. high and 3½ ft. in diameter at the base. Each pile is assumed to contain from 400 to

500 lb. The piles are arranged in rows approximately 12 ft. apart, the spacing between the piles being 6 ft. Mexican labor is found to be most satisfactory in gathering the salt, and each man is supposed to pile from 8 to 10 tons per day. Theoretically, the salt remains in piles until the brine has drained. As the mother liquor contains the majority of the impurities which might possibly contaminate the salt, this step is additional insurance of its purity. However, the salt does not drain freely, but remains damp for a considerable time. When the salt is gathered, a buggy of a special type is used. These vehicles are of galvanized iron and have a channel-like cross-section of body. Owing to the field being soft, the rear wheels are 20 in. in diameter and have a 12-in. face. The front wheels are 16 in. in diameter and have a 4½-in. face. The bed is 3 ft. wide, 8 ft. long, and 6 in. deep. The axles are attached rigidly to the body, so that curves are turned by slipping the front wheels. These buggies carry from 500 to 1 000 lb. of salt, and are hauled across the field by a ⅜-in. cable operated by a gasoline engine. At the dumping point the wagons are pulled up an inclined plane until they assume an angle of approximately 44°, at which slope the wet salt slides freely into a hopper above a car which has a body 30 in. wide, 4 ft. 6 in. long, and approximately 30 in. high. The bottom of this car is hoppered both ways at an angle of 45°, so that the sides, instead of the ends, are used for doors. These cars are operated on a double-track railway, about ½ mile long, by an endless rope haulage system operating at a speed of 200 ft. per min. A 10-h. p. gasoline engine will move seven loads and seven empties on these tracks. The cars dump into a boot serving a drag conveyor which elevates the salt to a point sufficiently high to command the 50-ton bins at the loading terminal. Here the tramway buckets are filled by a lever chute of the ordinary type. The buckets are dispatched from the loading terminal at the tap of an automatic spacing gong. The rate established is fifty-six buckets per hour.

The tramway has carried material continuously at the rate of 23 or 24 tons per hour, and is an unqualified success. The stations, with the exception of the summit, are not housed so as to permit readily of winter operation, but the line is designed so as to prevent snow and other weather conditions from interfering with its operation.

The line is not designed for passenger service, but the line operators use it in preference to going on horseback over the rugged

mountain trails. A ride over this line is a never-to-be-forgotten experience. Starting from either terminal, the passenger rises rapidly to the summit, with a change from the torrid heat of the valley to the chill of the mountain heights. The descent is a reversal of this condition, so that it is difficult to dress properly to be comfortable throughout the trip.

Acknowledgment is tendered to the following gentlemen, to whose individual foresight, skill, and untiring energy are due the design, erection, and initial operation of the tramway: Mr. White Smith, President, Mr. Will Smith, Secretary, W. H. Leffingwell, M. Am. Soc. C. E., Chief Engineer, Mr. Daniel Kuhnle, Superintendent, Mr. M. O. Bolser, Electrical Engineer, and Mr. Harry Hilderman, Foreman of Construction, respectively, of the Saline Valley Salt Company; Mr. J. W. Smith, Local Manager, Mr. S. S. Webber, Chief Engineer (retired 1914), Mr. William Hewitt, Mechanical Engineer, Mr. S. W. Benson, Chief Draftsman, and Messrs. C. H. Wickham, L. T. Hays, and T. J. Murphy, Field Engineers, respectively, of the Trenton Iron Company, and, later, Mr. R. H. Hall, Field Engineer, and Mr. George Hall, Foreman, for the American Steel and Wire Company.

DISCUSSION

Mr. Lamb.

RICHARD LAMB,* M. AM. SOC. C. E.—On studying this paper, with reference to other plants which have been built, it is seen that it describes novel features and conditions which were overcome with difficulty.

At first one might wonder that such an expensive plant should be constructed for handling so common and cheap a commodity as salt, for a 7½-lb. bag of the best grade retails at present for 10 cents, and lasts a family for quite a time. During normal times much salt is brought to the United States as ballast for vessels, and is sold at a low price.

Recently, the speaker was examining a cement stone deposit in Alabama, and found, at a point 3 miles from tide-water, a salt deposit from which most of the salt used by the Confederate States was mined during the Civil War. A comparison of the plant described by Mr. Carstarphen with a project to mine salt and deliver it by cableway to tide-water from the Alabama deposit indicates that the latter would have a great advantage. Later, the speaker visited the Gulf Coast of Louisiana, and there found that vast deposits of salt are now being mined and shipped by boat or train with little or no intermediate transportation, and the owners cannot sell all they can mine. However, it is presumed that the market was examined before the Saline Valley cable tramway was built.

One of the novel features of this tramway is its excessive gradients; the speaker knows of no other long cable tramway where they are so steep. The only other plant in the United States which can compare with it in length is that of the American Copper Company, in Wyoming, 16½ miles long. In the latter, however, the maximum vertical angle is about 20°, as compared with 40° in the Saline Valley plant.

The undertaking was bold, in that such gradients had not heretofore been overcome, and the success of the plant would depend on the efficient service of the grip used. The speaker would have thought that the Pohlig grip would have been successful; however, the author states that the plant was not equipped with either the Pohlig or the Bleichert grip.

In the Bleichert system the difficulty, on steep gradients, is that in going up the steepest incline of the cable, just before surmounting the saddle at the tower, the hauling cable is pulling somewhat downward, and the resultant force has a tendency to retard the carriage. In the system designed by the speaker, the hauling cable is kept parallel with the bearing cable, and the downward pull is practically

* New York City.

Mr. Lamb.

obviated. This feature is used on the line of the American Copper Company.

The speaker's discovery of this principle was due to the following incident. A patented logging cable tramway had been erected by parties who stated that it was practical. The bearing cable was carried on A-frame supports. A trolley carriage ran on the cable, and from it the log to be transported was suspended. Attached to the log was a cable which was made fast to a whippletree drawn by a horse. As the trolley approached the cable support, it had to ascend the incline caused by the inherent sag in the cable. This direction was one line of force in the parallelogram of forces. The direction of the pull by the horse was horizontal. The resultant force, consequently, was downward, and the trolley would not pass over the support. The project was a failure, and the difficulty could not be overcome under the conditions of the system.

If a trolley trackway is on a grade and a rope is attached to the trolley to pull it in a horizontal direction up grade, the trolley will not move. It is necessary to have the hauling and bearing cables as nearly parallel as possible. The steeper the gradient, the more difficult this problem becomes. Practical plants embodying this principle were described by the speaker some years ago in a paper* before this Society. An inherent feature of cable tramways is that they must be built in an absolutely straight line. In the logging cable tramway described by the speaker, in the paper referred to, the sheaves carrying the hauling cable were hung so as to permit the alignment to deviate from a straight line, but it had to be very nearly straight.

In the Saline Valley line there are three changes of direction, or four separate cable tramways joined by curved rail at the points of deflection, with electric motors to operate the hauling cable at each angle. At each control station the traction cable is made a unit for that station only, and is driven by its own motor. The summit station is arranged so that both the grip sheaves are on the same shaft, and are controlled by a motor which supplies the difference between the power required to hoist the ascending bucket and that developed by the descending bucket. The power is not transmitted around curves, as in surface cable-power street railroads.

The tension ordinarily used on cable tramways is about 30 000 lb. per sq. in. of cable. The cable can never be stretched to approach closely the horizontal; if so, it would be strained beyond its elastic limit. The towers are usually about 250 ft. apart. Tension stations are about 5 000 ft. apart. In some cases spans as long as 1 600 ft. between towers have been operated successfully.

* *Transactions*, Am. Soc. C. E., Vol. XXXII, p. 44.

Mr. Lamb.

The Chilcoot Pass cableway was built mainly for the transportation of passengers. For years this Pass had been traversed by miners and others seeking gold, and it was so difficult that many lives were lost there. The Trenton Iron Company built a cableway that carried freight and passengers successfuly over the Pass. It was intended to build the power plant on a plateau, named White Horse, near the center of the Pass, but an avalanche buried the site, and it was with difficulty that another could be found.

The power was furnished by a porcupine boiler in which the tubes were of such weights as could be carried to the site by Indians. The bearing cable could not be more than $\frac{5}{8}$ in. in diameter, as it also had to be transported by Indians. The transportation from Trenton to the final location was more expensive than the material of the plant. Great care had to be exercised in making the cable, as there was one span about 1 500 ft. long. The first person to traverse the line was a girl who used her canoe as a car.

Mr. Scholtz.

H. F. Scholtz,* Assoc. M. Am. Soc. C. E.—There are several places in the Pacific Northwest where tramways for transporting logs are in successful operation. The logs are taken out from places practically inaccessible by any other method. One instance is in the Cascade Mountains, near Index, Wash., where logs as large as 6 ft. in diameter are carried across a deep ravine and down a precipitous cliff to the mill.

Mr. Carstarphen.

F. C. Carstarphen,† M. Am. Soc. C. E. (by letter).—The writer appreciates the favorable tenor of the discussion which has been presented. Several points mentioned by Mr. Lamb are worthy of note. The first deals with the question of the price of salt, which no doubt was of great importance to the promoters of the Saline Valley Salt Company, but was only of passing interest to the builders of the tramway. Information as to the prices which can be obtained for salt on the Pacific Coast shows that they range from $3.50 to $40.00 per ton, depending on the grade and character. Ocean salt sells for about $3.50 per ton, which is somewhat less than the computed retail price of $26.70 per ton for salt, at the rate of 7½ lb. for 10 cents. It is the writer's understanding that milled salt suitable for table use sells wholesale at from $16 to $20 per ton, so that with the ample demand of the fisheries and other users of salt on the Pacific Coast and in the Northwest territories, the development of the Saline Valley salt field was considered by its promoters to be a worthy business enterprise.

Mention has been made of the length of the tramway built for the American Copper Company, of Grand Encampment, Wyo. The writer quotes from a report on file relative to this line:

* Carrollville, Wis.
† Trenton, N. J.

Mr. Carstarphen.

"The tram was built in four sections of about four miles to each section. The first section from the smelter to Station 1 was approximately four miles in length, with a raise of perhaps 4% across an almost level prairie. The buckets were transferred there from Section 1 to Section 2, etc. Section 2 was almost four miles, and the first three miles above Station 1 was a gradual raise of probably 4 per cent. For the last mile of Section 2 the raise was probably 12%, and crossed one or two deep gulches. Station 2 to Station 3 was over a rough broken country. It consisted of long spans and some very high towers. From Station 3 to the loading station the tramway crossed the Continental Divide. Station 3 was in a canyon, and the mine was also in a canyon. The tramway line raises probably 3 000 ft. between the two terminals along Section 3.

"This tramway is of the old Leschen lug type; one of the first four cable tramways built by that company. The lugs consist of a piece of sheet steel bent around the cable and a cast-iron button or lug riveted into this steel, and this button in turn entered into channel irons on the buckets by means of triggers and slide bars. There is one of the same type of tramways at the Sunnyside Mine, Silverton, Colo.

"The cables on the tramway at Encampment were of the 6 by 7 lang lay type, and probably $1\frac{5}{8}$ or $1\frac{1}{4}$ in. in diameter. The traction ropes were $\frac{3}{4}$ in. in diameter.

"Tram was driven by power generated from boilers located at the various stations along the line.

"Mr. Riblet was on the ground, and designed the structures and figured all the towers, angles, and deflections of the cables. Equipment was all furnished for this line by the A. Leschen and Sons Rope Company, of St. Louis, Mo.

"The traction rope on these towers was supported by small sheaves which hung on a cross-beam 21 in. from the top of the stationary cable. These sheaves were grooved, and when the buckets passed over the sheave the lug would raise the rope high enough to pass between the rope and the sheave. One main difficulty was this: These channel irons on the buckets were bolted to the frames with $\frac{1}{4}$-in. carriage bolts, two on each side, or four bolts altogether. The track ropes were so slack that the buckets in passing over the long spans would hang down so low that the traction rope would pull the lugs through these irons, breaking off the bolts and letting the buckets run away. The lug, being riveted to the cable, would immediately whirl over a few times and, as it passed, the next tower would hang on a rod that supported the timber for the tower sheave. Towers were all set on the ground without any anchorages, and consequently it dragged this tower to the next one and then hooked onto that one and pulled it to another one. Sometimes the writer would go up along the line and find as many as four towers all piled up in one bunch. The engines were powerful, and the man in charge had no idea when anything was hung up. They just kept pulling as long as they could move the thing.

"The downward pressure on the traction rope in coming over the Continental Divide, caused by the driving station and the loading station being in the canyon below, and long spans leading to each

one of them, was so great that it would simply cut through a new tower sheave in a very few hours. We experienced most of our difficulties in the operation of this tramway on the three miles of country adjacent to the Continental Divide." Mr. Carstarphen.

The topography, due to the erosion of the Bridger sandstones, is not comparable with the faulting and mountain-forming tendencies which have developed the extreme slopes and elevations characterizing the mountainous areas of California; and, therefore, the difficulties of design of the two tramways are not of the same order.

No difficulties were experienced on the Saline Valley tramway due to the faltering of the carriers in passing the saddles supporting the track cables, on account of the traction rope not being parallel with the bearing cable; nor is this tendency of any importance on tramways of the Trenton-Bleichert construction, for it is eliminated by the fundamental principles of tramway design. Reference to Fig. 6, which represents loaded and empty carriers passing an 80% grade, will show that the track cables and traction rope are practically parallel.

In modern tramway design, it is customary to place the cable supports so that the angle formed by the intersecting tangents developed by the chord slope and the deflection of the empty and loaded cable, is kept within a reasonable amount, depending on the radius of the saddle and its capacity in angular magnitude.

The tensions used in tramway track cables have changed from time to time. Twenty years ago track cables were erected with a tension as low as 14 500 lb. per sq. in. Tensions were then increased to 22 500, 25 000, and 30 000 lb. per sq. in. Beyond this stage increased knowledge was obtained of the stresses developed in locked coil cables due to bending. The influence of frictional hysteresis on the modulus of elasticity of the cable in bending was ascertained, so that the question of the proper tension of a tramway track cable depends largely on the condition of the particular tramway which is being designed.

If it is admitted that stress, due to bending, is developed in the outermost wire of a track cable, it is consistent to assume that a cable can be made to fail due to excessive bending only. It is well known that, if sufficient direct stress is developed in the cable, it can be pulled in two. Therefore, there is an economic tension to be applied to the track cable which will develop a minimum combined stress in the outermost wire due to direct load and bending.

Bending stress is developed in the track cable due to its support of the loaded carrier. The measure of this bending is a cable curve. The tangent of the intersection angle is equal to the weight of the load divided by the horizontal component of the tension in the track cable. If the loaded carriers are low in weight and the tension in the track

Mr. Carstarphen.

cable is high, the intersection angle of the curve under the carrier will be small; that is, the curve will have a long radius, and the stress developed in the outermost wire, due to bending, will be less than that due to direct tension.

This is the fundamental criterion which should be considered in designing aerial tramways, where it is desirable that a proper life of track cable should be obtained, together with a minimum of interference due to improper elevation of tower saddles.

The spacing of the towers depends on the tension in the track cables and the weight of the load supported. With a proper radius for the saddle supporting the track cables, the spacing of the towers depends on the economic construction height of these supports. The farther apart the towers are, the greater must be their height in order to balance the deflection and clearance required between the bottom of the carrier and the ground surface. The longest span which the American Steel and Wire Company has put in is 3 600 ft.

Tension stations may or may not be used, at the designer's option. In fact, it is customary to control the change of position assumed by the track cable when loaded and empty by erecting it as an anchored span. It is well known that the product of deflection and tension at any point on a suspended cable is constant. Accordingly, in anchored spans, if the loading changes, the deflection tends to remain a constant, due to the change in tension. If the cable were non-elastic and symmetrically loaded, there would be no change in deflection between the empty and loaded positions. In practice, there is a slight change in these positions, due to the elastic properties of the cable.

The pronounced limitation of tramway location to a straight line is no longer apropos, as present-day designers do not hesitate to handle moderate angles on the tramway supports. In this manner angles of considerable magnitude may be introduced into the alignment of the tramway without interfering seriously with its operation; nor is it necessary in cases of this sort to build a special structure called an "angle station."

Although the discussion has not been extensive, it has served a useful purpose in demonstrating that the proper design of aerial tramways is based on sound principles of engineering, and that such systems of transportation are not to be considered as expedients to be adopted when no other method can be used. Aerial tramways are safe, sane, and satisfactory.

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Paper No. 1395

THE WATER SUPPLY OF PARKERSBURG, W. VA.*

By William M. Hall, M. Am. Soc. C. E.

With Discussion by Messrs. John W. Hill, Alexander Potter, George W. Fuller, Nicholas S. Hill, Jr., Walter E. Spear, T. Kennard Thomson, H. F. Dunham, Theodore S. Johnson, Philip Burgess, James H. Fuertes, Morris Knowles and J. D. Stevenson, Edward Mayo Tolman, and William M. Hall.

Synopsis.

This paper describes the process of selecting a new water supply for the City of Parkersburg, W. Va., the investigation of the possible sources of supply, especially the natural underground supply in the neighboring Ohio River bottoms and plateau, the physical and geological phenomena relating thereto, the methods considered for collecting the water for pumping, and the novel method finally adopted, which consists of an infiltration system composed of strainer pipes laid in the bed of the river and overlaid with a bed of washed gravel and sand.

Introduction.

The City of Parkersburg, W. Va. (population 17 842 in 1910), constructed works for a public water supply in 1884. The pumping station was established on the banks of the Ohio River, and for more than 25 years the supply was drawn from the muddy and polluted waters of that stream, and used for all purposes without any treatment whatever. With increasing quantities of sewage entering the river from Pittsburgh, Wheeling, and other communities in the Ohio Valley

* Presented at the meeting of February 21st, 1917.

above Parkersburg, this supply became increasingly unsatisfactory. Typhoid fever was quite prevalent, and investigations in 1906 and 1907 indicated for that disease annual death rates of 75 and 60, respectively, per 100 000 population.

In 1907 a law was enacted creating for Parkersburg a Water-Works Commission, composed of three city officials, including the Mayor, and four citizens, of whom the writer was one.* This Commission instituted investigations for the purpose of determining the best method of treatment of the Ohio River water, if that should continue to be the supply, and whether or not water could be obtained from wells, cribs, or from a gravity source. As a part of the project, consideration was also given to a new pumping station to supersede the original plant, which had suffered much from wear and tear in pumping muddy water for such a long time. It is not within the scope of this paper to describe the details of the pumping station, distributing reservoirs, pipe lines, etc.

Preliminary Investigations.

Morris Knowles, M. Am. Soc. C. E., was engaged to make recommendations relative to a new water supply. In addition to gravity and impounding sources of supply and filtration of the Ohio River water, he investigated the feasibility of obtaining a well-water supply from the Camden Farm, which lies immediately north of the upper city limits of Parkersburg. This is shown by Fig. 1.

Mr. Knowles reported in August, 1908, for a recommended 4 000 000-gal. daily supply, that, although the well proposition was attractive, further investigation should be made, perhaps in other localities, or somewhat removed from the Camden Farm. His test of a 12-in. well did not indicate a serious lowering of the ground-water level, but, for safety, a more elaborate test was recommended, as it was realized that a large factor of safety should be required in establishing a well plant. Based on his recommendations, the citizens voted on April 22d, 1909, to authorize a bond issue of $270 000 to build the proposed new water-works; and he was authorized to prepare plans and specifications for the construction thereof.

* The Commission consisted of Messrs. W. D. Pedigo, Mayor, C. D. Forrer, of the Board of Affairs, Walter Gerwig, Councilman, and the following citizen members: Messrs. S. D. Camden, H. H. Moss, B. S. Pope, and the writer. Mr. Camden was elected President of the Commission, and Mr. Forrer, Secretary. After the expiration of Mr. Forrer's term of office, Mr. George P. Chase, Attorney, though not a member of the Commission, served as Secretary.

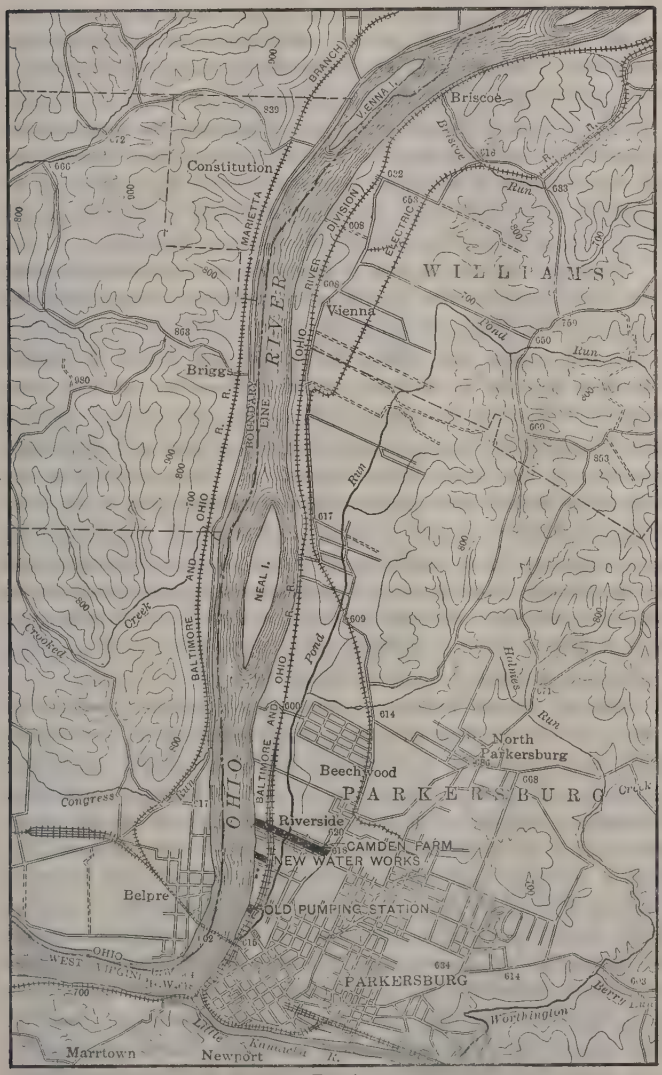

Briscoe
Constitution
Briggs
Vienna
WILLIAMS
Pond Run
Pond Run
Pond Run
Hatmes Run
North Parkersburg
Beechwood
PARKERSBURG
Riverside
CAMDEN FARM
NEW WATER WORKS
OLD PUMPING STATION
Belpre
PARKERSBURG
Worthington
Marrtown
Newport
OHIO RIVER
NEAL I.
BALTIMORE AND OHIO R. R.
BOUNDARY LINE
MARIETTA (BRANCH)
VIENNA I.
ELECTRIC (DIVISION)
Crooked Creek
Congress Run
Little Kanawha R.
Berry Run
Creek

FIG. 1.

Some doubt as to the permanency of the well-water supply arose in the minds of some of the city officials, and, on request, further consideration was given to the matter in July, 1909, by Mr. Knowles, who called in Mr. F. G. Clapp, a geologist, as advisor on this subject. Mr. Clapp favored the well-water supply, but was also aware of the possibility of having to build new wells in future years, thus making it wise to secure at the outset plenty of land in the vicinity of the projected plant; such additional land would also protect the purity of the supply by avoiding surface pollution near the wells.

There was still one member of the Water-Works Commission who was unable to agree with the otherwise unanimous opinion as to the adequacy of the proposed well supply and its suitability as to quality. To obtain further data, James H. Fuertes and George W. Fuller, Members, Am. Soc. C. E., were engaged in November, 1909, to investigate and report on the proposed well supply, with a view to settling the differences of opinion between the city officials and the citizen members of the Water-Works Commission. They investigated the well-water project with considerable thoroughness, and found that, though it would be possible to obtain a 4 000 000-gal. supply from the gravel and sand layers underlying the bottom-lands in the neighborhood of the Camden Farm, it would be necessary that such a well system should extend along the river for at least 4 000 ft., thus making its cost greater than that of a filtered-water supply from the river. They recommended the adoption of the latter.

The city officials passed an ordinance authorizing the Board of Affairs to build the filtration works and appurtenances, but this authorization was nullified by an injunction suit which stopped the further expenditure of funds, made available by the bond issue for the purpose, without the approval of the Water-Works Commission. In the meantime Mr. L. E. Smith interested some of the leading citizens in a system of water supply consisting of a manifold of pipes laid in a sand bar in the Ohio River. Samuel M. Gray, M. Am. Soc. C. E., was then engaged to report on the advisability of adopting the so-called Smith system. His report, delivered on May 26th, 1910, being favorable, the works were built under his direction, and for 5 years or more the water has been clear, clean, and of satisfactory sanitary quality. Not only has the new water supply been satisfactory to the citizens, but it is gratifying to note that the new pumping equipment has effected

an economy sufficient to pay the capital charges on the bond issue required by the new plant.

It is believed that the development of the new supply has placed on record many data of general interest to the Profession with regard to ground-water; and that this is so with respect to the investigation of the well-water supply by Messrs. Knowles, Fuertes, and Fuller, and in particular in the case of the somewhat novel system built by Mr. Smith under the direction of Mr. Gray. In placing before the Society the principal features of the Parkersburg water project, the writer feels of necessity bound to make liberal use of the reports of the above-mentioned gentlemen, and wishes to state that much of the following is an abstract compiled from their several reports in the preparation of which he is indebted to them.

Geological Conditions.

The geological conditions of the Ohio Valley in the vicinity of Parkersburg were investigated at length by Mr. Knowles with the aid of Mr. F. G. Clapp.

Attention is first directed to the cross-sectional profile of the Ohio River at Lock and Dam No. 18 (Fig. 2), furnished by the writer from investigations made under his direction.

The bottom-lands on the West Virginia bank of the Ohio River, for a distance of 5 or 6 miles north of and above Parkersburg, contain more or less water-bearing strata. This is shown by numerous wells in successful use for domestic and manufacturing purposes within that tract, and is also confirmed by an examination of the materials removed in drilling these wells.

From a study of the history of the Ohio River in this immediate locality, it appears that the river bed has shifted its position materially along the part south of the promontory at Briscoe, where the rock of the West Virginia shore extends down to and beneath the river bed. At present the Ohio lies well to the west in the river bottoms, in some places nearly reaching the foot of the hills forming the rugged upland of the Ohio shore. In earlier days the river unquestionably skirted the West Virginia foot-hills. The result of this shifting has been the deposition of a substantial quantity of gravel and sand in the present bottom-lands, with a stratum of sandy gravel extending north from above the upper limits of the City of Parkersburg for about 5 miles, and having a varying width averaging perhaps ¾ mile.

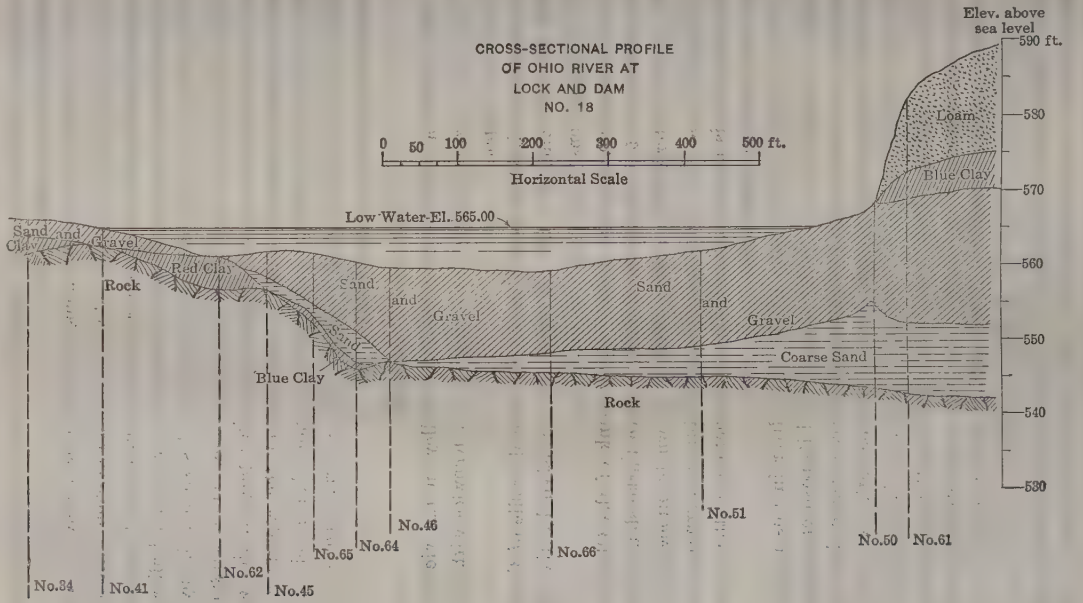

CROSS-SECTIONAL PROFILE
OF OHIO RIVER AT
LOCK AND DAM
NO. 18
0 50 100 200 300 400 500 ft.
Horizontal Scale
Elev. above sea level
590 ft.
580
570
560
550
540
530
Loam
Blue Clay
Low Water El. 565.00
Sand and Gravel
Red Clay
Rock
Sand and Gravel
Sand and Gravel
Coarse Sand
Blue Clay
Rock
No.34
No.41
No.62
No.45
No.65
No.64
No.46
No.66
No.51
No.50
No.61

FIG. 2.

This stratum varies in depth from perhaps 15 to 25 ft., and averages approximately 20 ft., according to available information. It is overlaid with a stratum of loam ranging in depth from 25 to 40 ft. The bottom of the sand and gravel stratum varies in elevation roughly from about 540 to 550 ft. above sea level, and its top surface from about 560 to 570 ft.

A more detailed statement of the local geology, with comments on its significance with reference to the ground-water supply of Parkersburg, is set forth with various comments by Mr. Clapp in the following paragraphs.

Local Geological Details.—In order to make sure of the adequacy of the present and future supply, the formation of the valley and its sediments was carefully considered. The real bottom of the Ohio Valley is not the present surface of the river or of the first and second bottoms, but is the bed-rock surface, or surface of the solid rock (sandstone and shale) underlying these. Along the part of the valley considered, this bed-rock surface lies from 50 to 125 ft. or more below the first bottoms, as determined by the depth of numerous wells and test borings.

The origin of the buried bed-rock surface is due to erosion at a time—thousands of years ago—before the overlying gravel filling existed, when the whole country stood higher above the ocean level, and hence the present bed-rock was at the surface of the ground. The deeper parts of the bed-rock valley (*i. e.*, those now buried more than 100 ft. below the present bottoms) were eroded by the ancient Ohio River as it cut its way deeper into the surface of the country while it was comparatively high, forming a sort of gorge below the general level of the bed-rock surface in the valley. Thus there were (and still are) three rock levels: (1) that of the gorge, now from 100 to 125 ft. below the present first bottoms; (2) that of the ordinary bed-rock in the valley, now from 50 to 90 ft. below the first bottoms; and (3) that of the surrounding hills, the tops of which stand from 200 to 300 ft. above the bottoms. The sand, gravel, and clay filling which forms the bottoms and higher unconsolidated terraces along the valley was formed at a more recent date than the rock valleys, due to various incidents in connection with the Ice Age, during which the northern part of the territory now occupied by the State of Ohio

was covered with a great glacier, damming the river valleys at various places.

For the most part, the buried bed-rock surface of the Ohio Valley is rather flat, and, under the Camden Farm and elsewhere, it lies from 50 to 90 ft. below the lower bottoms. On this rock bench rest the water-bearing gravels, from 20 to 30 ft. thick, from which it is proposed to draw the water supply by pumping from wells. The gravels are covered by clay, fine sand, and loam, reaching to the surface of the bottom-land. Many household wells driven into the gravels, and also geological inferences, have shown that the formation is similar and continues for miles up and down the main valley.

No part of the rock gorge underlies the Camden Farm, but at this place it is below the present river. Presumably it meanders from side to side of the valley, and farther up and down stream it may underlie the gravels, as it does at the test wells on the land of the Parkersburg Iron and Steel Company. In order to show that this deeper gorge will not interfere with the supply of the wells by draining off the water at a lower level, it may be stated that the deeper gorge is filled for the most part with finer material, such as fine sand and clay, and these do not hold large volumes of water. Moreover, the portion of the underflow of the river which follows the gorge is separated from the water in the upper gravels by layers of finer, more impervious material, which tend to keep the two bodies of water separate.

Another geological feature which needs consideration is the terrace known as the "sand plains region", lying back of Boreman Hill, and standing at an average elevation of 80 ft. above the first bottoms, or 130 ft. above the underlying bed-rock. The sand plains are about ½ mile wide, and extend from the bottom-lands of the Ohio Valley at Beechwood for ½ mile to Worthington Creek. It is a geological fact, based on good evidence, derived from what is known of similar sand plain regions at various places along the Ohio, that this strip was the ancient valley of Worthington Creek, or the Little Kanawha River. The depth of the sand and gravel filling there is unknown, but it is considerable; and through shallow wells which have been drilled there, it is known that it contains a large volume of water. This water is supplied by the rain which falls on the sand plains and flows westward, through the underlying sand and gravel, to the Ohio Valley,

where a large part of it passes through the gravels underlying the Camden Farm.

Thus there will be two sources of supply for the wells: the ground-water from the sand plain district, and that from the bottom-lands in the main valley of the Ohio, both bodies of water moving toward the wells. At no other locality in the vicinity could this double supply be obtained in such great quantity.

Gravel-Water versus Rock-Water.—The question arises: Can a better or more adequate supply of ground-water be obtained by drilling to some lower level than is planned, that is, into the bed-rock? To this question the reply is decisively "no". The rocks underlying the river valleys and the entire vicinity of Parkersburg are similar in character to those in the surrounding hills, and consist of shales and sandstones of the Dunkard formation (upper barren coal measures), which contain very little water, and nowhere in these rocks could a sufficient supply be obtained. In fact, a supply as large as is necessary does not seem to exist anywhere in the shales and sandstones of the coal measures, or in the thin limestones scattered through them. Many hundred feet below the surface of the valley, a large enough supply could be found in limestones, but, even if it was practicable to sink wells to that depth, the water would be found to be salty and unfit for use.

Estimated Yield of the Wells.—Calculations by Mr. Clapp agreed with those by Mr. Knowles, that fourteen wells will yield from 3 000 000 to 4 000 000 gal. of water per day, which is more than is at present needed, and will suffice for some years to come. However, after from 8 to 12 years, it may be necessary to add a new well every year or two for several years, in order to keep pace with the increase in population and to replace possible deteriorating wells. Hence, plenty of land should be secured in the vicinity of the wells. Owing to the character of the water, and the known type of river gravels, the wells may last 15 years or more without the necessity of being replaced. In order that the supply may be maintained at approximately its initial quantity, the wells should be cleaned occasionally, and, if pumped with an air-lift, the air pressure should be reversed frequently, in order to blow out any fine material which may collect in the pores of the gravel close to the wells or in the strainers.

Arrangement of the Wells.—The water in the gravels is moving in two directions. A part of it, derived from the rainfall on the bottom-lands farther up the valley, is moving slowly down stream, approximately parallel to the Ohio River. A second part of the gravel-water, derived from the rainfall on the sand plains, is moving from them toward the river, in general at right angles to it. Therefore, the proposed arrangement of the wells in two lines, one parallel to the river and the other at right angles thereto, is the best for securing all available water. The wells should be at least 300 ft. apart.

Hydraulics of the Local Well-Water Field.

So much discussion having arisen in Parkersburg as to the relative merits of a well-water supply and a filtered-water supply from the Ohio River when Messrs. Fuertes and Fuller made their examination in the fall of 1909, it was determined to conduct pumping tests for a period of at least 3 weeks. Accordingly, a deep-well pump was set up in one of the 12-in. wells sunk by Mr. Knowles. This pump was operated under the observation of James R. McClintock, M. Am. Soc. C. E., from January 18th to February 8th, 1910, and during this period quantities ranging from 480 000 to 575 000 gal. daily were pumped. Eight special observation wells were driven, and frequent studies were made of the water level therein, and also in thirty-one existing driven wells within 3 or 4 miles up the river on the Camden Farm site, including two wells in the sand plains east of the bottom-lands.

Water levels in the river were taken several times a day, and samples of the well water were collected frequently for analysis. The temperature of the well water was uniformly between 51 and 52° Fahr., and that of the river water during the period under observation ranged from 34 to 38° Fahr.

Summary of Results of Studies of Ground-Water Supply, as to Source, Quality, and Quantity.

The results of the pumping tests are given in a report by Messrs. Fuertes and Fuller, dated February 21st, 1910, with perhaps unusual clearness, and a summary of this information is given at some length, as follows.

Water-bearing Strata.—The first essential in securing a ground-water supply was to ascertain the conditions with respect to the water-bearing strata, as it is obvious that without such strata a ground-water supply for the city was out of the question. Reference has already been made to the local geological conditions, and these may be further spoken of from the standpoint of water supply as to the character of the water-bearing strata.

The elevation of the Ohio River opposite the Camden Farm ranges from about 562 at low water to about 621 for extreme high water, which floods the bottom-lands to a depth of from 10 to 20 ft. At low water the depth of the river varies somewhat, but approximates 5 ft., according to the surveys made by the United States Government.

The present river bed opposite the low lands in question is made up largely of sand and gravel. The proportion of gravel and the range in gravel size vary considerably. The proportion of sand apparently increases with the distance from the natural dam at Briscoe.

Excellent building sand for local construction work is secured from the river bed by dredging at a point opposite the Camden Farm. Samples of this sand, after removing the gravel by screening, show what would be called a medium river sand, having an "effective size" of about 0.28 mm. Test wells sunk by Mr. Knowles, and from which samples of material were placed on exhibition in the rooms of the Water Commission, show a substantial proportion of relatively fine or medium-sized gravel from which the sand was largely eliminated in the process of removal from the ground. So far as it has been possible to ascertain, these bottom-lands, at a depth of about 40 ft. below the surface, contain a sand and gravel layer with the sand predominating. The sand is of medium size, filling the voids of the gravel and thus controlling the resistance of the stratum to the flow of water. The average depth of the layer is about 20 ft., and not more than one-fourth of it is above extreme low water. In a rough way, it appears that during dry years the water in the river is below the level of the surface of the sand and gravel stratum for about 4 months, and sometimes it is 3 or 4 ft. below for weeks at a time.

Deep wells show that, beneath this water-bearing stratum of sand and gravel, there are layers of shale and clay, rather irregularly spaced, and interspersed with more or less sand and gravel. So far as it has been possible to ascertain, however, the sand and gravel layers below

the main deposit are too thin and too variable as to continuity to make them worth serious consideration in connection with a municipal water supply.

The general conditions as to the present river bed at Lock and Dam 18, about 2 miles below Briscoe, are shown on the sectional profile, Fig. 2.

Source of Ground-Water.—The second step in studying a ground-water project was to ascertain the source of the water found in the water-bearing strata, and particularly the source and direction of the flow of water in such strata in the event that a municipal supply were to be taken regularly therefrom.

Inspection of local conditions shows at once that the promontory at Briscoe, with the rock extending across the river a short distance beneath the river bed, clearly precludes the possibility of any substantial quantity of ground-water coming from points above Briscoe and passing longitudinally down the bottom-lands. Accordingly, a water entering the sand and gravel strata must come either from the river, by infiltration through the bottom and sides of the river-bed, or from the rainfall on the bottom-lands themselves or the upland district immediately tributary thereto.

Inspection shows that these bottom-lands are traversed longitudinally by Pond Run for the greater portion of the distance, and at the upper or northern end there is another run which passes diagonally across the bottom-lands and enters the Ohio at Vienna. These surface streams have a drainage area of some 5 sq. miles back in the hills. The hilly sections contain much clay and loam, and are generally of an impervious character, as distinguished from the so-called sand plains. Pond Run apparently has an almost impervious bed. Its slope is exceedingly light, and the fine materials deposited show that, except at times perhaps of heavy rainfall, no appreciable quantity of water passes from this run into the water-bearing strata below.

Along the edge of the upland area there are a number of flowing wells and wet places, showing clearly that the water entering the ground in the hilly upland district meets impervious strata as it reaches the edge of the bottom-lands, and that it is easier for it to force its way to the surface than to enter the sand and gravel strata.

Of course, some rainfall on the bottom-lands themselves would reach the underlying strata of sand and gravel, but the fine and impervious

nature of the 40-ft. layer of loam precludes this from reaching any substantial volume. It must also be borne in mind that the loam stratum is flooded with muddy Ohio River water from time to time, and the surface at such periods is covered with a deposit of fine silt and clay.

During wet weather, perhaps when the water in the Ohio is high, it would appear that the water contained in the pores of the extensive sand and gravel layers is largely from the river, but to some extent it is from upland sources, the high river stage damming up some of the upland water.

Messrs. Fuertes and Fuller were firmly of the opinion that, during low-water stages in the Ohio, the water which can be drawn from the strata is that which has been stored within the pores of the material at times of previous high water. They were also certain that, during low stages in the river, as this stored water is removed or naturally flows away, whatever water enters these sub-surface strata of sand and gravel is largely of river origin. They concluded, therefore, that, as a source of supply for a municipality, the water from such strata, at the end of protracted droughts, must come from the Ohio through the bottom or sides of the present river bed along the bottom-lands below Briscoe.

Disposition of Ground-Water.—The next point on which it was necessary to make inquiry, in studying the feasibility of a ground-water supply, was the answer to the question: Where does the water entering the porous sand and gravel layers in question now go? The answer is shown clearly by the studies which have been made and the profiles of the water levels secured from different wells prior to the occurrence of high water during the winter of 1909-10, as well as in the study of various water levels or contours during and subsequent to the January rise in the Ohio. Unquestionably, the water in the porous sand and gravel layers, found at some distance below the bottom-land, ultimately makes its way more or less irregularly and slowly into the Ohio.

Several features of interest and importance were disclosed in studying the question of the relation between the passage of the water from the Ohio into what is practically a large underground storage reservoir, as above described, the water storage therein, and

also the passage of water therefrom back into the river. This is indicated by the diagrams, Figs. 3 to 10.

In the first place, it was found that the water in the porous sand and gravel layers did not rise to such high levels as those found in the river itself. This is indicated by the readings in test wells almost at the bank of the river; and shows that, even during fairly high velocities in the river, there is still so much mud and silt deposited on its sides and bottom that a considerable head is required to overcome the friction sufficiently to allow substantial quantities of the water to pass through this thin and more or less impervious deposit.

In the second place, in discussing the question of the disposition of the water stored in these sand and gravel layers, it is necessary to consider the character of the material, and especially the finer particles of sand making up the water-bearing stratum or underground storage reservoir. In this regard the observations showed that considerable head was required in order to cause the water to flow in substantial quantities through the sand interspersed with gravel. For instance, it was found that the river water scarcely affected the level of the water in wells 2 000 ft. from the river, notwithstanding that in the river itself there was a 20-ft. rise during the period in which the observations were made.

This shows two important features: one is that this underground reservoir fills from the river only to a slight extent during floods of short duration; the other is that considerable water cannot be removed at a single point without creating quite a steep slope on which the water will flow to the point of removal. This means that the water stored in this underground reservoir can by no means be removed entirely; in fact, in order to remove a substantial portion, it would be necessary to establish points of withdrawal at quite frequent intervals throughout the area in question.

Pertaining, however, to the answer to the main proposition set forth under this heading, it may be said that the water stored in this sand and gravel layer during floods makes its exit into a falling river with quite steep slopes when the quantity stored is great. Considering these underground sand and gravel layers as a storage reservoir, therefore, it is seen that, after having once been filled during a protracted flood, the reservoir, in the absence of any pumping, slowly becomes emptied as the river falls. Here it may be pointed out that, although

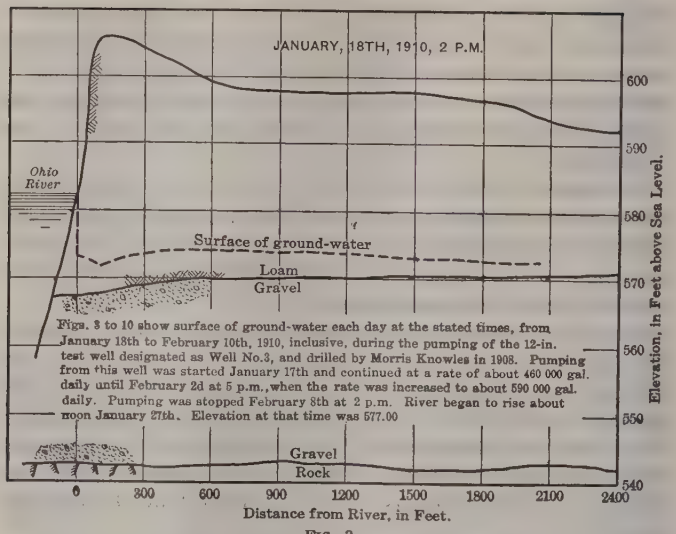
the mud deposits on the bottom and sides of the river bed are of much significance in controlling the rate at which this underground reservoir will fill, they seem to be of less significance as regards its emptying, on account of the lifting of the deposits from the sides of the stream bed by the pressure of the ground-water in its flow to the river.

Collection of Ground-Water.—The next step in considering a ground-water proposition, particularly one like that at Parkersburg, where a flowing underground stream does not exist, is to ascertain the best means of drawing or collecting the water from the porous sand and gravel layers containing it.

It is true that during the pumping tests the bottom-lands were so saturated with water that the withdrawal of 500 000 gal. or more per day from a single 12-in. well made very little impression on the surrounding water levels. A cone of depression was noticeable, particularly during the latter portion of the pumping period, but the information obtained therefrom was slight when compared with that available from other sources. This is indicated by the diagrams, Figs. 3 to 10.

The studies by Messrs. Fuertes and Fuller showed clearly that during the pumping period, from January 17th to February 5th, 1910, the ground-water contours were nearly parallel to the river. In other words, there was substantially no difference in the elevation of the ground-water at approximately equal distances back from the river throughout the entire length of the bottom-lands. This fact tends to show that the quality and porosity of the underlying porous strata of these lands are not materially different throughout their length. It also shows that there is no substantial flow of water longitudinally through the porous strata forming the underground reservoir in question. Therefore, they concluded that it would not be judicious to provide for the removal of ground-water by a line of wells at right angles to the river bank. This is shown by Figs. 3 to 10.

The records obtained by Mr. Knowles in December, 1908, show that, even when less than 350 000 gal. per 24 hours were pumped from a 12-in. well, the water level was lowered in the well about 13 ft. without lowering the surface materially at points less than 200 ft. from the well. Furthermore, this cone of depression had its apex below the surface of the rock on which the sand and gravel strata rest.



Figs. 3 to 10 show surface of ground-water each day at the stated times, from January 18th to February 10th, 1910, inclusive, during the pumping of the 12-in. test well designated as Well No.3, and drilled by Morris Knowles in 1908. Pumping from this well was started January 17th and continued at a rate of about 460 000 gal. daily until February 2d at 5 p.m., when the rate was increased to about 590 000 gal. daily. Pumping was stopped February 8th at 2 p.m. River began to rise about noon January 27th. Elevation at that time was 577.00

FIG. 3.

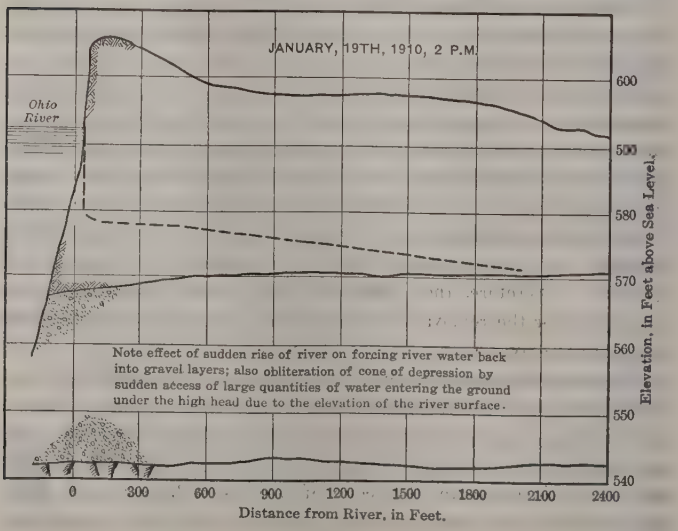


FIG. 4.

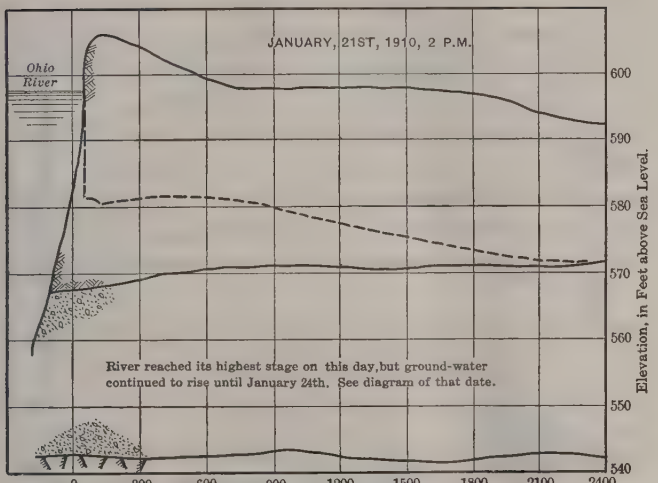

JANUARY, 21ST, 1910, 2 P.M.
Ohio River
River reached its highest stage on this day, but ground-water continued to rise until January 24th. See diagram of that date.
600
590
580
570
560
550
540
Elevation, in Feet above Sea Level.
0
300
600
900
1200
1500
1800
2100
2400
Distance from River, in Feet.

FIG. 5.

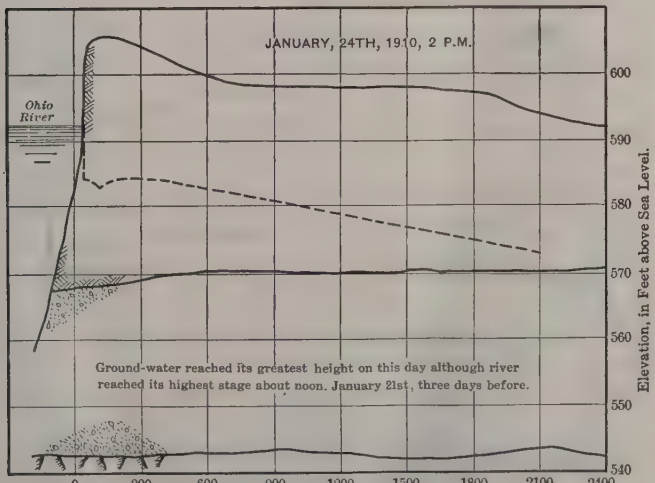

JANUARY, 24TH, 1910, 2 P.M.
Ohio River
Ground-water reached its greatest height on this day although river reached its highest stage about noon, January 21st, three days before.
600
590
580
570
560
550
540
Elevation, in Feet above Sea Level.
0
300
600
900
1200
1500
1800
2100
2400
Distance from River, in Feet.

FIG. 6.

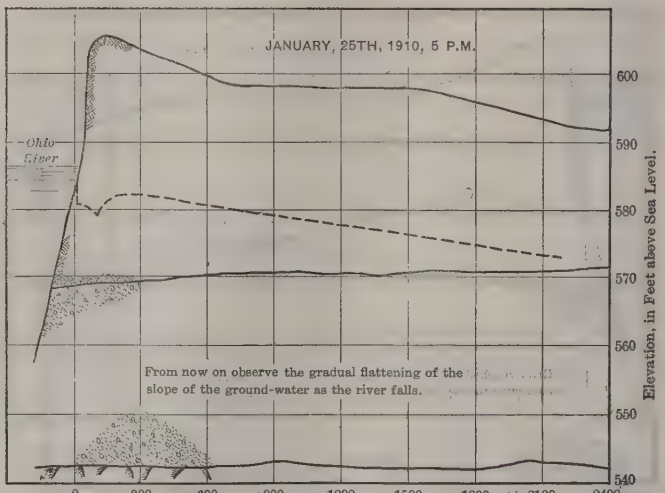

JANUARY, 25TH, 1910, 5 P.M.
Ohio River
From now on observe the gradual flattening of the slope of the ground-water as the river falls.
600
590
580
570
560
550
540
Elevation, in Feet above Sea Level.
0
300
600
900
1200
1500
1800
2100
2400
Distance from River, in Feet.

FIG. 7.

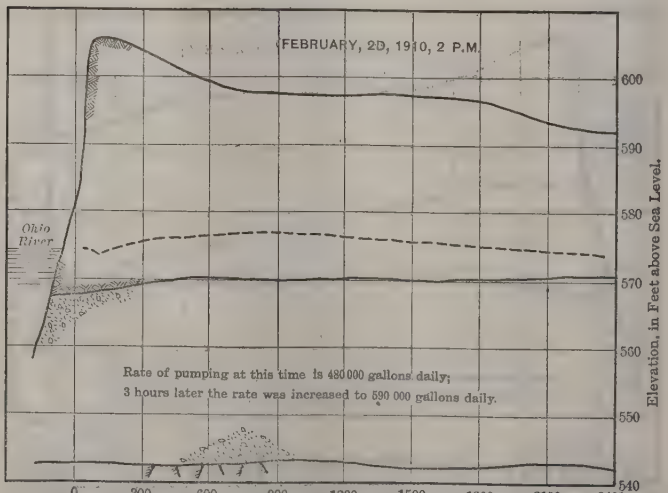

FEBRUARY, 2D, 1910, 2 P.M.
Ohio River
Rate of pumping at this time is 480 000 gallons daily;
3 hours later the rate was increased to 590 000 gallons daily.
600
590
580
570
560
550
540
Elevation, in Feet above Sea Level.
0
300
600
900
1200
1500
1800
2100
2400
Distance from River, in Feet.

FIG. 8.

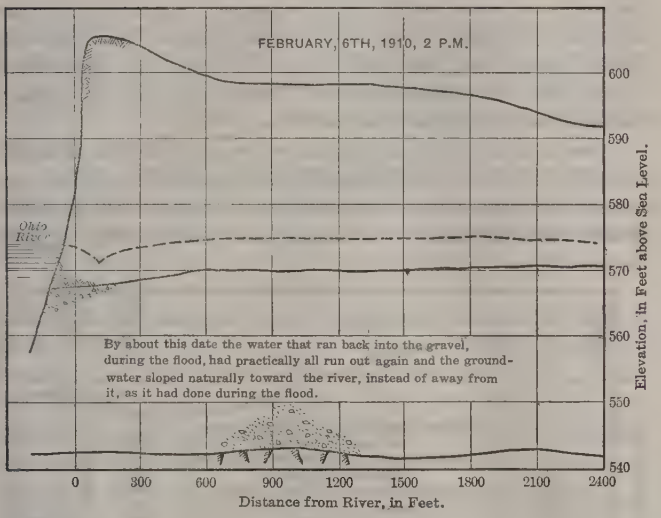

FEBRUARY, 6TH, 1910, 2 P.M.
Ohio River
By about this date the water that ran back into the gravel, during the flood, had practically all run out again and the ground-water sloped naturally toward the river, instead of away from it, as it had done during the flood.
600
590
580
570
560
550
540
Elevation, in Feet above Sea Level.
0
300
600
900
1200
1500
1800
2100
2400
Distance from River, in Feet.

FIG. 9.

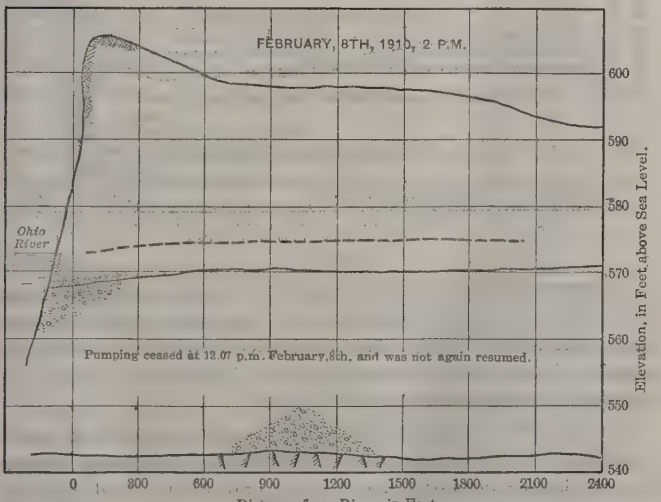

FEBRUARY, 8TH, 1910, 2 P.M.
Ohio River
Pumping ceased at 12.07 p.m. February 8th, and was not again resumed.
600
590
580
570
560
550
540
Elevation, in Feet above Sea Level.
0
300
600
900
1200
1500
1800
2100
2400
Distance from River, in Feet.

FIG. 10.

The volume of water which could be pumped from this well at the end of this protracted period of low water, therefore, was limited by the flow which could reach the bottom of the sand and gravel strata at any given location for the removal of water; at the point in question the limit seemed to be less than 500 000 gal. daily.

These data led to the conclusion that it would be less advantageous to attempt to place an underground collecting gallery at or near the bottom of the porous strata in question than to collect water by a series of wells, assuming that the investment cost would be kept at about the same figure.

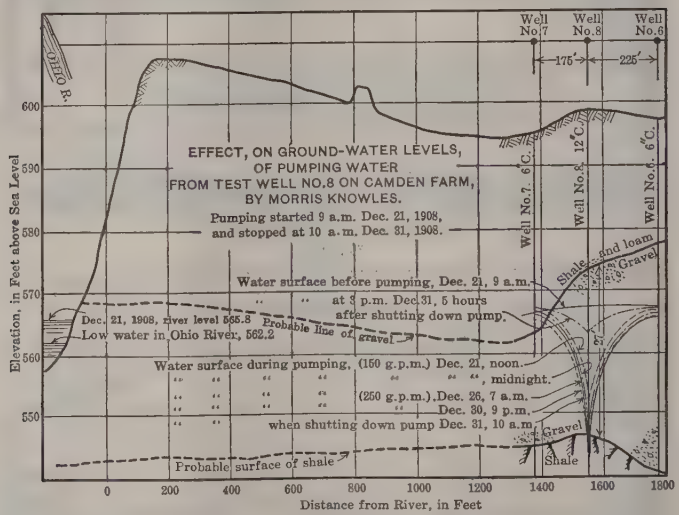


FIG. 11.

Taking everything into account, and considering only the geological conditions, it is perfectly clear that the best method of collecting an underground water supply from the district in question would be by a series of wells, each not less than 10 in. in diameter, parallel with the river, and approximately 400 ft. apart.

Mr. Knowles' recommendation for a line also perpendicular to the river was influenced by the lay of the land which was being given to the city by the Senator Camden estate provided the land was used for water-works purposes.

Furthermore, as a safeguard during protracted periods of drought, it was the opinion of Messrs. Fuertes and Fuller that it would be wise to sink these wells in a line parallel to and about 200 ft. from the bank of the river, rather than at a greater distance, and not at right angles thereto.

Available Quantity of Ground-Water.—The investigations of Messrs. Fuertes and Fuller showed clearly that, during high and moderate stages of the Ohio, a supply of ground-water ample for all needs of the city exists in the porous sand and gravel layers in the bottom-lands above the Camden Farm.

It was also their opinion that it was reasonably certain that, at the end of low-water periods, a well-designed system of wells would provide for the needs of the city for some time to come.

To obtain a supply of 4 000 000 gal. daily from wells, it would be necessary to provide at least ten wells of the size and positions above mentioned; and further, in order to insure this volume of water at the end of protracted periods of drought, it would be essential to provide a plant for increasing artificially the quantity of water filtering in from the river, through its bottom and sides, to the wells in question. Messrs. Fuertes and Fuller considered it possible to secure an increase of infiltration by means which are noted subsequently in describing the feasible methods of securing a ground-water supply under local conditions.

Quality of Ground-Water.—During the Fuertes and Fuller investigations particular attention was paid to the quality of the water, both as shown by the results of analyses of samples made during Mr. Knowles' investigations, and also of analyses of samples taken during the pumping tests and analyzed jointly by Professor Merriam, of Marietta, and by Mr. J. R. McClintock, who served as Resident Engineer during these tests. In brief, these analyses showed a water which was of excellent appearance, and satisfactory in every way for domestic and manufacturing uses. There was a slight tinge of turbidity in the well water when examined in 1-gal. bottles, but it was so slight that it would not be noticed in a tumbler. At times of increasing suddenly the rate of pumping, or on starting up the pump after it had been shut down for a few minutes, slight increases in the turbidity were observed, but these quickly disappeared.

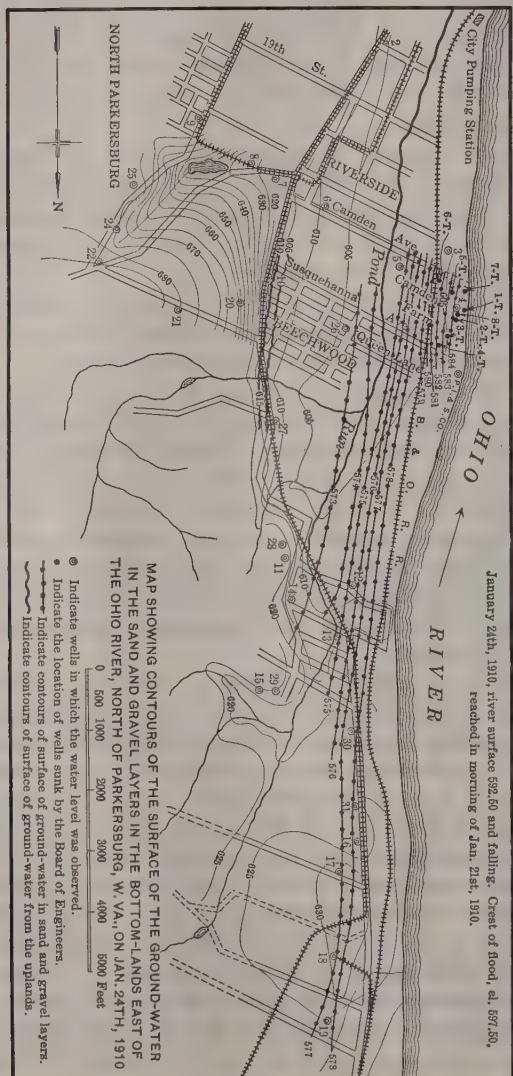

City Pumping Station
19th St.
RIVERSIDE
Camden Ave.
Camden Farm
Pond Ave.
Susquehanna
BEECHWOOD
Queen Lane
Run
OHIO RIVER
NORTH PARKERSBURG
N
January 24th, 1910, river surface 592.50 and falling. Crest of flood, el. 597.50, reached in morning of Jan. 21st, 1910.
MAP SHOWING CONTOURS OF THE SURFACE OF THE GROUND-WATER IN THE SAND AND GRAVEL LAYERS IN THE BOTTOM-LANDS EAST OF THE OHIO RIVER, NORTH OF PARKERSBURG, W. VA., ON JAN. 24TH, 1910
0 500 1000 2000 3000 4000 5000 Feet
Indicate wells in which the water level was observed.
Indicate the location of wells sunk by the Board of Engineers.
Indicate contours of surface of ground-water in sand and gravel layers.
Indicate contours of surface of ground-water from the uplands.

FIG. 12.

All samples analyzed were tested, without any effort to eliminate the slight turbidity mentioned.

Briefly stated, the bacteria in the well water ranged from 14 to 170, and averaged 63 per cu. cm. The total hardness of the well water ranged from 76 to 98, and averaged 90 parts per million. The river water samples collected on the same days showed a total hardness ranging from 52 to 71, and averaged 63 parts per million. Corresponding figures for wells in the Beechwood district were 88, 102, and 94, respectively.

Samples of well water taken from the test well were analyzed carefully for iron, and found to contain about 0.5 part per million. There was no sign of this iron precipitating out, and experience elsewhere shows that, with the quantity of iron above stated, difficulties are not encountered.

Samples of water taken from test wells contained rather objectionable quantities of iron in some instances, evidently due to the water dissolving iron during long intervals of standing in contact with the metal.

Slightly increased quantities of iron were found in the water of the well at the steel works, but not to an objectionable degree. This greater quantity of iron is explained partly by the flow of water through a long line of pipe and partly by the connection which the well has with underground porous strata where the water remains in contact with pervious layers for many months and perhaps years.

Messrs. Fuertes and Fuller, taking everything into consideration, had no hesitancy in stating that the quality of the water from the porous sand and gravel layers of the bottom-lands at and above the Camden Farm is thoroughly satisfactory for domestic and manufacturing purposes. They placed the water from such wells in the same general class as filtered water obtained from a properly built and well operated filter plant treating water from the Ohio River in the vicinity of Parkersburg.

There are some slight differences in the quality of the water from wells and from filters, the principal feature of which would be the hardness on account of the wells yielding water coming largely from the Ohio River at times when it is in flood and fairly soft. Messrs. Fuertes and Fuller did not attach much importance to this difference, because ultimately, during low-water periods, the well water would

also have its source in the Ohio. So far as quality is concerned, they put waters from filters and from wells in the vicinity of the Camden Farm on a parity. However, the writer is somewhat partial to the natural water when equally free from bacteria and other objectionable qualities.

MOST PRACTICABLE METHOD OF SECURING A GROUND-WATER SUPPLY.

Messrs. Fuertes and Fuller discussed and considered from various viewpoints the most practicable method of securing a ground-water supply, and their conclusions may be briefly summed up as follows.

It is not feasible to secure a municipal water supply for the City of Parkersburg by galleries or wells running at right angles to the Ohio River.

Wells of very large diameter, or tunnels, would not be a fruitful investment, on account of the steep slopes of the cones of depression which would be formed around the points of withdrawal of water. Small wells, not less than 12 in. in diameter, afford by far the best way of collecting the ground-water supply from the porous sand and gravel layers in the bottom-lands at and above the Camden Farm.

These wells should be in a line parallel to the river, about 400 ft. apart, and about 200 ft. from the bank.

A deep tunnel for receiving the flow of the wells is far less expedient than a force main receiving the waters from individual wells, due to the expense of tunneling at the depths required, and other local conditions.

DESCRIPTION OF WELL-WATER SYSTEM AS CONSIDERED BY MESSRS. FUERTES AND FULLER.

The studies of the plant necessary to carry out most economically and advantageously the methods just described of securing a ground-water supply from the locality in question, are briefly described as follows.

Extent.—Ten wells should be established, in a line extending along the river front, approximately 200 ft. from the bank, and 400 ft. apart.

Location.—As the well layout would cover about 4 000 ft., it is not feasible to place all the wells on the Camden Farm, or, in fact, all below the Steel Works, as the porous underground strata do not extend a sufficient distance down stream. It is assumed, therefore, that the well plant will extend to above the Parkersburg Iron and

Steel Works. Prudence, however, would suggest that they should not reach above the lower end of Neal's Island, on account of the shallowness of the channel east of that island, the absence of scouring velocities therein, and the imperviousness of the stream bed.

Wells.—It is contemplated to sink shafts, 7½ ft. in diameter, brick lined, and 40 ft. deep, or down to the top of the porous sand and gravel layer. In each there should be a cased well, 12 in. in diameter, with a brass strainer for a length of at least 20 ft. at the lower end. This 12-in. pipe would be connected with a centrifugal pump placed at the bottom of the 7½-ft. shaft, the masonry wall of which would be extended up to above extreme high water, some 12 ft. above the ground level in this locality.

Comparison of Well-Water Supply with Filtered Supply from the Ohio River.

In the report of Messrs. Fuertes and Fuller comparative estimates of cost were given, both for the well-water supply and filtered river-water supply to provide an average daily yield of 4 000 000 gal. and be capable of supplying water for short periods at a maximum rate of 6 000 000 gal. per 24 hours.

In substance, this report stated, on the basis of figures detailed therein, that the cost of construction and of operation of a first-class well system and a first-class filter system at the Camden Farm site were substantially on a parity as to the quality of water and as to the cost, both of construction and of operation, and that either plant could be depended on to supply such quantity of water as would be needed by the City of Parkersburg for some years to come. Mention is to be made of the fact, with reference to the well-water supply, that the estimates of cost include a suitable charge for land required to insure the adequacy of the system.

Further, the report stated that, by placing the filter plant nearer the site of the present pumping station, the investment cost could be made less for the filter plant, but with no substantial difference in operating and maintenance expenses between either plant at either site. The report finally recommended that all interests of the city would be best served by the establishment of a filter plant, near the site of the then existing water-works pumping station, to supply the city with properly filtered Ohio River water.

Smith Infiltration System.

An injunction suit, as already stated, prevented the city officials from acting independently of the full Water-Works Commission in building the proposed filtration system, as above recommended. Then, at the request of various prominent citizens of Parkersburg, the Smith system came up for consideration.

In May, 1910, Mr. Samuel M. Gray, reported on water-works improvements with special reference to the so-called Smith strainer-pipe system of filtration, his conclusions in brief being as follows:

1.—This system, under local conditions, is both feasible and practicable for furnishing an ample supply of water of suitable quality for the present needs of the city.

2.—If properly built and operated, this system will furnish water of a better quality than a mechanical filter plant, as regards steam-raising purposes, on account of the slight increase in permanent hardness which is caused by the use of a coagulant.

3.—Though higher in first cost than a mechanical filter plant, the operating cost of the Smith system would be materially less, amply so, in fact, to warrant the higher first cost.

4.—That water from the proposed driven wells would be inferior in quality to that from the Smith system, and, further, that such wells would be very likely to lessen in yield in course of time, and the water grow harder and contain more iron, with also the possibility of local contamination from surface sources.

On considering this matter further, Mr. Gray modified the detailed plans by recommending that the infiltration system should be divided into five units, with a separate suction pipe for each unit. The advantage of this was to facilitate back-flushing of the sand overlying the respective units of pipe manifold systems, and for making repairs and renewals without affecting seriously the service to the city. This recommendation was authorized, and, for the sake of economy, it was decided to abandon the so-called Camden Farm site and build a new pumping station on the river bank nearest the site selected for the infiltration system in order to economize in the cost of suction pipe. The relative locations are shown on Fig. 1.

Mr. Gray suggested using a greater length of slightly smaller strainer tubes, in order to increase the strainer surface. He also pre-

pared the plans and specifications for the construction of the infiltration system, as well as for other improvements to the water-works plant.

Contracts were then let for the infiltration system, and the work progressed to completion under the direction of Mr. Gray and according to the plans and specifications prepared by him. Mr. W. G. Wheelock was Resident Engineer.

Unusual Features of the Contract for the Smith Infiltration System.

As there had been considerable skepticism regarding the successful working of the Smith system, no similar works of comparable size having been in operation, the contract was made unusually exacting. Only 45% of the value of work done was paid on monthly estimates, the remainder being retained until 30 days after the test period of 364 days after the beginning of successful operation, as guaranteed. The contractor made the following guaranties for the system:

1.—The capacity to be 170 000 gal. per hour;
2.—The supply to be satisfactory, to the Water-Works Commission and the Board of Affairs, for domestic and industrial uses;
3.—The water to be of pleasing appearance, practically clear and colorless, and to contain not more than 5 parts per million of turbidity;
4.—With 3 000 bacteria per cu. cm. in the raw water (or higher), the filtered water to contain not more than 3% of that number; with less than 3 000 per cu. cm. in the raw water, the filtered water to contain not more than 100 per cu. cm.; counts to be on agar plates incubated at 20° cent., for 48 hours;
5.—Filtered water to contain on an average not more than 30 parts per million of total hardness in excess of Ohio River water;
6.—Iron in solution not to exceed 0.7 part per million;
7.—The foregoing requirements to be shown by daily tests covering period of 364 days from the date of beginning operation.

In case of failure to comply with the guaranty, the city was under no obligation to pay the retained 55%, and the completed plant was to become the property of the city.

The contractor was required to maintain the plant in good condition and thorough repair until acceptance.

CONSTRUCTION DETAILS OF THE SMITH INFILTRATION SYSTEM.

Under the contract, three units were completed and put in service on November 12th, 1911. The other two units were completed and put in service on December 15th, 1912. The plant has been in continuous operation ever since, except for 4 days during a flood which broke the former highest record by 5 ft., and thereby partly submerged the boilers and engines.

Fig. 13 shows the arrangement of the pipes in the infiltration system, and illustrates the appearance of the units while being laid.

The site of the plant is on the lower end of the bar at the foot of Vienna Island, at a point where the natural top of the bar forming the bed of the river is about 1 ft. below the low-water level. By inquiry, and the examination of Government hydrographic maps made about 10 years previous to the preparation of the plans, it was concluded that this bar had been practically stable for many years.

The system of five units of filters was placed by using two cofferdams of the ordinary box type common on the Ohio River lock and dam construction; and the suction pipes to the power-house were laid in an open trench, with the excavated material banked on each side. The work was done during the low-water seasons of 1911 and 1912, with the top of the coffer from 7 to 8 ft. above low water.

At this point the fluctuation of the water is about 58 ft., and stages up to 40 ft. are of annual occurrence. Coffer No. 1, enclosing Units 1 to 3, was built and removed in 1911, and Coffer No. 2 in 1912.

In the unwatered coffer the excavation for the pipes was made to 5½ ft. below low water. On the floor thus formed a 6-in. layer of gravel was placed, on which the pipes were laid and covered with 12 in. of gravel; on the gravel a 3½-ft. layer of clean washed sand was laid, bringing the top of the sand to within about 6 in. of the low-water line, or a few inches higher than the natural surface of the bar.

The infiltration system of five units thus formed is 110 ft. wide and 656 ft. long (1.65 acres), and is 5 ft. deep. The gravel was the washed and screened Ohio River product which passed a 1½-in. mesh, and with the sand removed. The sand was taken from the Ohio River and washed, and is fairly sharp and coarse. The filter

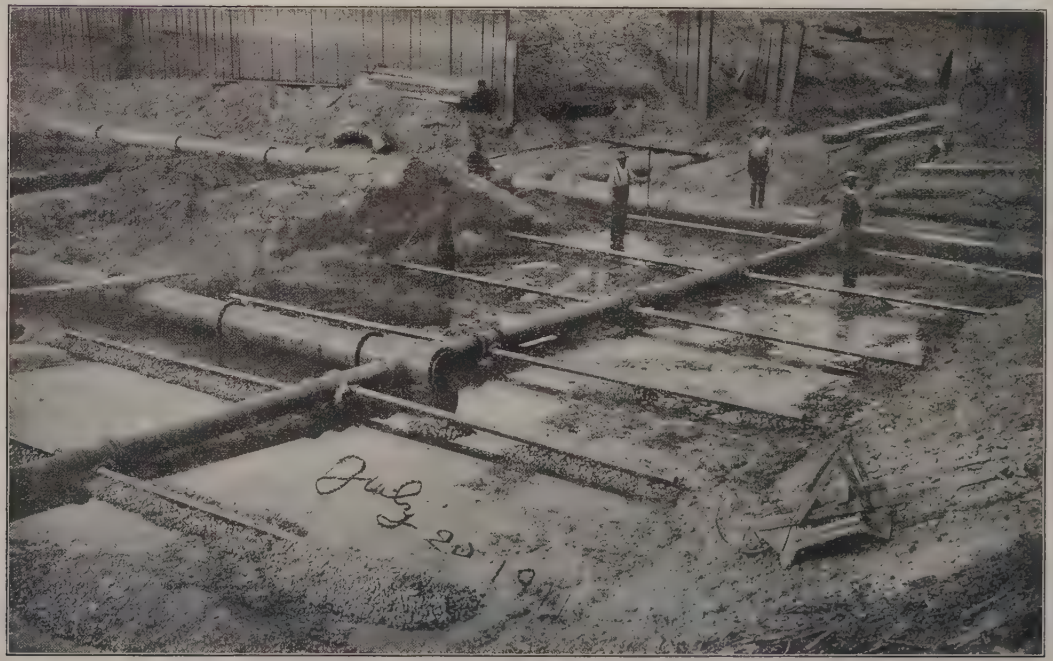
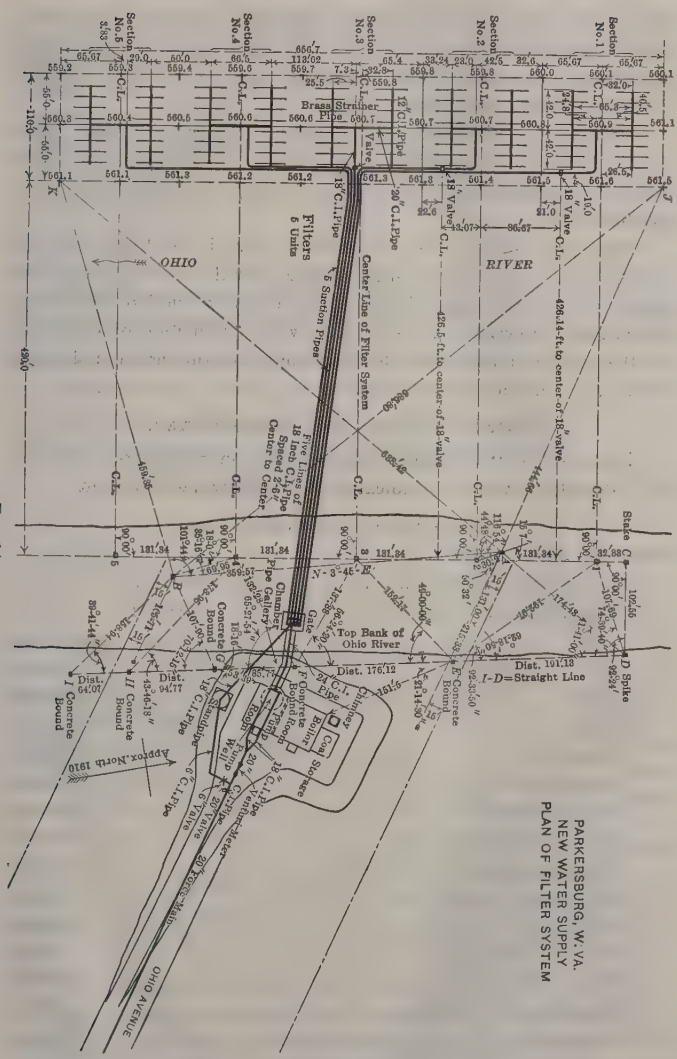


FIG. 13.—SOUTH HALF OF SECTION NO. 3, PARKERSBURG, W. VA., WATER SUPPLY.



PARKERSBURG, W. VA.
NEW WATER SUPPLY
PLAN OF FILTER SYSTEM

FIG. 14.

thus constructed lies in a natural sand bed of great dimensions, as described previously. It is the writer's opinion that a very large percentage of the water drawn is taken from the sand bed, possibly a larger part than that from the open river.

Each unit of the filter system contains 32 brass strainer pipes, 16 ft. long and 5 in. in diameter, or 160 pipes in the five units. Each pipe is perforated with 9 400 V-shaped slots, $1\frac{3}{8}$ in. long and about $\frac{1}{70}$ in. wide.

From each unit to the gate-chamber there is an 18-in. cast-iron suction pipe, each of these pipes forming a small reservoir. They are connected with the pumps by one 24-in. cast-iron suction pipe. This arrangement prevents serious trouble by the production of a vacuum in the suction pipes, with the resulting flow through the filtering material at an undesirably rapid rate.

Cost of the Smith Infiltration System.

The cost of the complete water-works improvements, including the infiltration system, is indicated in the following statement:

Water-Works Construction Fund.

Contracts Nos. 1 and 2:		
New mains including laying	$45 626.15	
Connecting service pipes to new lines	2 883.91	
		$48 510.06
Valves and hydrants		5 936.53
Pumps		32 442.30
Boilers		8 055.30
Pump well		21 556.67
Pumping station		11 441.32
Brick chimney		1 374.43
Laying pipe (force main)		5 021.27
Pipe and castings (force main)		15 987.01
Valves, etc. (not covered by No. 3)		2 394.63
Infiltration system		80 700.03
Venturi meter		772.10
Electric light plant		1 953.81
Plumbing and heating		555.72
Stand-pipe		2 029.65
Carried forward		$238 730.83

Brought forward		$238 730.83
Pumping station lot	$1 700.00	
Expense, sale of bonds	9 131.30	
Tests for water supply	2 435.20	
Refund to water-works fund, on account of preliminary engineer's report, test wells, etc	5 000.00	
		18 266.50
Amount paid to engineers:		
Preliminary reports and well tests		10 419.39
Design and supervision of construction		11 801.92
Miscellaneous expenses, Water-Works Commission, Not chargeable to above items:		
Office rent and expenses, clerk, Dr. L. O. Rose testing water, electric wiring to pumping station, and miscellaneous expenses		4 952.58
		$284 171.22
Cash balance, January 31st, 1914		2 658.76
		$286 829.98

The cost of operation is indicated by the following:

For the fiscal year, July 1st, 1912, to June 30th, 1913:	
Salaries and wages at pumping station	$5 541.00
Fuel at pumping station	3 319.61
Oil, packing, etc., at pumping station	889.76
Repairs and improvements at pumping station	2 142.09
	$11 892.46
Average pumpage per day	2 955 000 gal.
Total pumpage for year	1 078 675 000 gal.
Cost	$11.03 per million gallons.

The records for 1914 and 1915 show a slight reduction in this cost.

The expense of operating the filters in 1915, including flushing the five units 73 times, is estimated by the Superintendent's office as $673. The average quantity of water used for each unit at each

flushing is 40 000 gal. Although the tank capacity is 60 000 gal., it is the custom, in operation, to use 40 000 gal. at each flushing. The cleaning and flushing is practically all the expense which has been incurred in excess of pumping out of the open river. The time required to pass the 40 000 gal. through a filter is from 15 to 20 min. When pumping out of the open river with the old plant, some trouble was caused by ice, leaves, etc., but it is believed that the cost of pumping from the filters and keeping them cleaned is no greater than that of pumping from the open river.

The maximum pumped in one week in 1915 averaged 3 760 000 gal. per day and 154 166 gal. per hour. For 2 weeks after the great flood of 1913, the average pumpage and consumption exceeded 5 500 000 gal. per day.

The water has shown a little turbidity about three times, at river stages of about 45 ft. The writer has observed turbidity in that drawn from his service pipes only once.

From May 1st, 1913, to January 31st, 1914, daily tests of the water were made under the direction of the Commission by Dr. L. O. Rose in his laboratory at Parkersburg, and report thereof is shown in part in Appendix I. This report shows that the quality of the water produced by the filters conformed in every respect with the terms of the contract.

From time to time since making this series of daily tests, Dr. Rose has made tests of the water, and the data obtained by him for the calender year, 1915, are included in the Appendix. The water has been tested by numerous others, but the writer did not, until the summer of 1916, hear of a report indicating that it is not as good as required under the contract.

A recent report of it, prepared for the City of Wheeling, W. Va., has questioned its being of the highest standard. Since March, 1916, samples of water have been shipped monthly to the West Virginia State Bacteriologist, at Morgantown, W. Va., for test. Of about 14 shipments, several, when tested after standing a day or more, have been reported "unsafe". From February 15th to October 27th, 1916, Dr. Rose tested 38 samples and found *Bacillus coli* present in 11, and absent in all the others. For each of these 11, 2 tests were made of 1 cu. cm. and 10 cu. cm. each. In 3 of the samples *coli* were present in both tests, and in the other 7, it was absent in the 1 cu. cm. Of the other 27

samples, the two tests were made for 19 of them and only the smaller quantity for the other 8.

On account of these reports, early in September, 1916, Mr. Gray was engaged to have all the filter beds examined and cleaned. He assigned G. H. Leland, M. Am. Soc. C. E., to carry out this work personally, and Mr. Leland was engaged in doing this for nearly a week. The writer spent several hours with him on his second day of inspection. At several places there were holes in the sand and gravel covering. The holes were from 5 to 10 ft. in diameter, and were formed like craters. They extended down to within 1 or 2 ft. of the level of the brass strainer pipes. The work done under Mr. Leland's supervision consisted of stirring the deposit on top of the filter beds with a powerful force pump, at the same time giving each section numerous back-flushings, and thereafter filling the holes with fresh gravel and sand. In the 10 samples tested by Dr. Rose since that work was done, no *coli* were found. Of these 4 were tested in both 1 cu. cm. and 10 cu. cm., as previously explained, and the other 6 in only the smaller quantity.

The writer is of the opinion that tests should be made at regular intervals, weekly or more frequently; but, on account of the confidence in its quality, it appears that such an important work to safeguard the health of the community has not been done regularly.

Under date of June 10th, 1913, Mr. I. L. Birner, Bacteriologist, certified to Contractor Smith, after a series of tests in a laboratory at the pumping station, that:

"An examination of the results of 400 separate analyses of the filtered water at the Parkersburg filtration plant, made between January 23d and June 4th, 1913, covering a widely varying set of conditions, from a river stage at 5.8 to 58.9 ft., and with a river turbidity varying between the extreme limits of 15 and 1 950 parts per million, shows a clear water, with an average total hardness of 10.5 parts per million in excess of the river water; the total iron in solution, of the filtered water, was 0.19 of one part per million, against an average of 5 to 16 parts in the river, and a bacterial removal, or bacterial efficiency, of 98.28 per cent."

Until recent years an accurate count of typhoid cases in the city has not been available. However, the public impression is that, previous to 1912, the yearly number was comparatively very large.

For 1915 there were twenty-two cases which started in the city and ten cases which were brought in after illness commenced, making thirty-two cases in all. Of the twenty-two, three were traced to a well, three were railroad employees, and one was a United States clerk, and these men were out of the city much of their time.

Dr. Stone, the City Health Officer in 1914, informed the writer that all the typhoid cases in the city that year except four were traced definitely to other causes than the city water.

Dr. Rose has stated that not one recent case of typhoid has been proved to have come from the city water supply, and this report is confirmed by Mr. Nearn, City Health Officer, whose present duty includes an investigation of the cause and history of all such cases. However, the monthly reports of typhoid cases in 1916 show a little higher numbers than for 1915.

Thus far, the system has been entirely satisfactory to the citizens. Its success has been fully equal to the most sanguine hopes of the writer when, as a commissioner, he voted for its adoption. However, he realizes that the experimental stage is not past.

Three units are now ending their fifth year of service, and the other two their fourth, with comparatively little indication of failure in quantity, or in the quality of the water.

However, the writer knows that it is only a question of years when the sand and gravel over the strainer pipes will have to be replaced by fresh clean gravel and sand; the principal question being the number of years before this will become necessary in order to maintain the good quality of the water. He is also impressed with the great importance of vigilance and care in its operation, and of systematic daily or weekly tests of the water.

APPENDIX I

Report of Tests of the Water from the Filtration Plant and the Ohio River, for January, 1914.

By Lonzo O. Rose, M. D., Parkersburg, W. Va.

The results are given in parts per million.

	Date.	Turbidity.	Alkalinity.	Incrustants.	Total hardness.	Iron.	Bacteria per cubic centimeter.	Rate of pumping.	Vacuum.	Stage of river, in feet above low water.
R.	Jan. 1	70	14	55.0	69.0	3.26	600			
F.	" 1	Clear	26	57.5	83.5	0.02	13	2 200	14.5	10.1
R.	" 2	50	21	60.0	81.0	2.70	200			
F.	" 2	Clear	37	60.0	97.0	0.01	4	2 200	17.0	9.2
R.	" 3	50	16	60.0	76.0	2.30	300			
F.	" 3	Clear	30	55.0	85.0	0.1	8	2 300	17.0	8.4
R.	" 4	20	16	57.5	73.5	2.22	200			
F.	" 4	Clear	30	55.0	85.0	0.08	3	2 000	16.5	8.2
R.	" 5	20	16	67.5	83.5	2.23	500			
F.	" 5	Clear	31	60.0	91.0	0.12	12	2 550	19.0	8.7
R.	" 6	15	11	52.5	63.5	1.18	200			
F.	" 6	Clear	32	57.5	89.5	0.09	5	2 900	18.0	9.0
R.	" 7	15	15	62.5	77.5	1.60	800			
F.	" 7	Clear	31	60.0	91.0	0.06	7	2 200	18.0	9.0
R.	" 8	12	14	72.5	86.5	1.80	800			
F.	" 8	Clear	31	62.5	93.5	0.06	9	2 450	19.0	8.9
R.	" 9	15	18	67.5	80.5	2.04	900			
F.	" 9	Clear	30	62.5	92.5	0.09	12	2 300	18.0	9.9
R.	" 10	50	18	70.0	83.0	2.52	1 200			
F.	" 10	Clear	32	67.5	99.5	0.07	22	2 450	17.0	12.0
R.	" 11	150	15	50.0	65.0	4.6	3 100			
F.	" 11	Clear	31	57.5	88.5	0.1	16	1 900	17.0	12.8
R.	" 12	140	16	60.0	76.0	3.00	1 500			
F.	" 12	Clear	34	55.0	89.0	0.11	14	2 600	17.0	14.4
R.	" 13	100	9	75.0	84.0	6.50	1 700			
F.	" 18	Clear	14	70.0	84.0	0.12	14	2 400	17.5	15.4
R.	" 14	100	9	75.0	84.0	5.06	1 900			
F.	" 14	Clear	27	67.5	94.5	0.06	5	2 100	15.0	12.9
R.	" 15	70	9	55.0	64.0	4.60	500			
F.	" 15	Clear	24	65.0	89.0	0.11	16	2 200	18.5	10.6
R.	" 16	50	12	55.0	67.0	3.20	600			
F.	" 16	Clear	17	50.0	67.0	0.1	24	2 700	8.0	9.4
R.	" 17	50	12	55.0	67.0	2.52	1 100			
F.	" 17	Clear	20	60.0	80.0	0.11	11	2 500	8.5	9.2
R.	" 18	40	16	62.5	78.5	3.00	1 200			
F.	" 18	Clear	22	60.0	82.0	0.15	23	2 000	8.0	8.8
R.	" 19	30	16	57.5	73.5	2.92	100			
F.	" 19	Clear	25	60.0	85.0	0.1	18	2 300	8.5	9.2
R.	" 20	15	14	65.0	79.0	1.06	1 300			
F.	" 20	Clear	22	70.0	92.0	0.06	10	2 100	9.5	9.2

R. Indicates river water. F. Indicates filter water.

REPORT OF TESTS OF THE WATER FROM THE FILTRATION PLANT AND THE OHIO RIVER, FOR JANUARY, 1914.—(*Continued.*)

	Date.	Turbidity.	Alkalinity.	Incrustants.	Total hardness.	Iron.	Bacteria per cubic centimeter.	Rate of pumping.	Vacuum.	Stage of river, in feet above low water.
R.	Jan. 21	10	11	62.5	73.5	2.32	300			
F.	" 21	Clear	22	67.5	89.5	0.08	5	2 250	8.0	10.0
R.	" 22	25	14	57.5	71.5	3.52	1 500			
F.	" 22	Clear	22	60.0	82.0	0.11	5	2 250	8.0	11.5
R.	" 23	20	19	60.0	79.0	3.20	1 300			
F.	" 23	Clear	26	67.5	98.5	0.1	1	2 250	6.5	13.0
R.	" 24	100	11	72.5	83.5	7.04	1 900			
F.	" 24	Clear	26	70.0	96.0	0.1	6	2 500	4.0	16.0
R.	" 25	100	9	55.0	64.0	6.16	900			
F.	" 25	Clear	22	57.5	79.5	0.19	4	1 900	5.0	15.8
R.	" 26	90	10	52.5	62.5	4.00	400			
F.	" 26	Clear	24	47.5	71.5	0.15	6	2 150	7.0	16.1
R.	" 27	60	16	60.0	76.0	2.80	600			
F.	" 27	Clear	27	55.0	82.0	0.11	4	2 100	9.0	16.0
R.	" 28	60	13	60.0	73.0	3.62	500			
F.	" 28	Clear	25	65.0	90.0	0.07	7	2 300	8.0	16.0
R.	" 29	60	11	60.0	71.0	3.50	300			
F.	" 29	Clear	25	55.0	80.0	0.07	4	2 650	9.0	15.5
R.	" 30	60	12	52.5	64.5	4.00	300			
F.	" 30	Clear	24	57.5	81.5	0.05	6	2 500	7.5	15.9
R.	" 31	80	11	57.5	68.5	4.96	1 200			
F.	" 31	Clear	24	60.0	84.0	0.14	6	2 200	4.0	18.1

R. Indicates river water. F. Indicates filter water.

APPENDIX II

Reports of Bacteriological Tests of Water of the Parkersburg Water-Works for the Year 1915.

By Lonzo O. Rose, M. D.

Date of collection, 1915.	Source.	Bacteria per cubic centimeter.	*Bacillus coli* on following sowings:		
			1/10 c. c.	1 c. c.	10 c. c.
Jan. 23....	River	604	—	—	+
" 23....	Filter	3	—	—	+
" 24....	River	800	—	+	+
" 24....	Filter	3	—	—	—
" 25....	River	500	+	+	+
" 25....	Filter	5	—	—	+
" 26....	River	200	—	+	+
" 26....	Filter	2	—	—	—
" 27....	River	300	+	+	+
" 27....	Filter	4	—	—	—
" 29....	River	200	—	+	+
" 29....	Filter	2	—	—	—
Feb. 1....	River	1 200	+	+	+
" 1....	Filter	12	—	—	—
" 27....	River	200	—	—	+
" 27....	Filter	12	—	—	—
" 27....	Tap	18	—	—	—
May 4....	Pump Station	Sterile	—	—	—
" 4....	Parkersburg Machine Co	3	—	—	—
Aug. 19....	No. 1800 20th St.	2 500 dead end	—	—	—
Sept. 9....	No. 1800 20th St.	11 " "	+	+	+
Oct. 10....	No. 1303 St. Mary's Ave	10	—	—	—
" 10....	23d St. and Dickel Ave	44	—	—	—
" 10....	No. 569 5th St.	7	—	—	—
" 28....	No. 1711 Park St.	18	—	—	—
Nov. 1....	No. 712 Grafton St.	86	—	—	—
Dec. 20....	No. 409 12th St.	2	—	—	—
" 20....	No. 932 Juliana St.	4	—	—	—

+ = present.
— = absent.

DISCUSSION

Mr. J. W. Hill.

JOHN W. HILL,* M. AM. SOC. C. E. (by letter).—The writer has read with interest this paper on the recent improvement of the Parkersburg Water-Works. Having had the honor of making the original plans for the Parkersburg Water-Works Company in 1881, and later of revising these plans and constructing the original water-works in 1884, he is naturally more or less familiar with the location and history of this improvement.

The original works, planned by a company of which the late Senator J. N. Camden was a member and the leading spirit in the enterprise, looked almost exclusively to fire protection for the city and the furnishing of water to railroads and manufacturing enterprises; in fact, at that time, 36 years ago, little attention was paid to the hygienic qualities of water supplies in America.

The Ohio River water in its then polluted condition was accepted for drinking purposes by all the cities taking water therefrom. The dangers lurking in sewage were not then well known, and in some respects not suspected. Time, research, and experience have since demonstrated that the Ohio River water in its natural state has not been suitable for drinking purposes for many years, if it ever was.

The Smith system of filtration has been proposed for several cities along the Ohio River and came prominently to the writer's notice while he was the Engineer Member of the Ohio State Board of Health, when it was proposed as a mode of supply for the City of Portsmouth, a few years ago. At that time, as well as before, the proposition was studied and classed as an artificial sand and gravel filter, and it was thought that such a filter, placed beneath the bed of the river—or rather below low-water stage in the river—beyond reach and examination, during ordinary and high stages, would not satisfy the modern requirements of a filtered water supply.

Plain sand filters, such as are used at Philadelphia, Pittsburgh, Washington, and other places in the United States, and used extensively abroad, are constructed so that they may be under constant observation as to the performance of each particular unit, and when the loss of head has reached a maximum—or other conditions suggest—they can be taken out of service, the sand beds can be scraped and rehabilitated, and the filters restored to their original condition and efficiency. The Smith filters, however, can be examined and repaired only at long intervals, and, if it should occur that the quality of the water obtained from them was unsatisfactory during a period when the stage in the river was considerably higher than the sand bars in which the filters are placed, there would be no relief from the unsatisfactory quality

* Cincinnati, Ohio.

Mr. J. W. Hill

of the water except that which might be obtained from back-flushing the collecting pipes, and that, in the writer's opinion, is always a doubtful method of restoring filters to normal condition.

It was also thought, in connection with the Portsmouth proposition, that, in view of experience with water purification systems somewhat similar to the Smith filter along the Ohio River, there was no assurance, at any time, that the quality of the water would satisfy strict sanitary requirements, and, for this reason, objection was then made by the Ohio State Board of Health to the adoption of this system by Ohio cities.

At several places in the paper Mr. Hall mentions a possible source of water supply to wells from the Ohio River, on the assumption that the water will pass through the silted bed of the river and reach the wells through the sand and gravel in which the latter are driven.

From long experience, with wells along the Ohio River and elsewhere, it is the writer's conviction that wells or galleries sunk below the bed of the adjacent river seldom, if ever, receive water from it. Many tests for hardness, chlorine, and iron, in water from the wells and from the river or stream, have always shown a distinct difference, and, in the writer's experience, this has been sufficient to indicate that the water from the wells is not naturally filtered river water. This experience has been so uniform over a wide range of territory, and throughout a long period of time, that it has led him to conclude that water obtained from wells or infiltration galleries along a stream is always land-water or ground-water percolating through, or pocketed in, the pervious strata, some of which in due time reaches the stream in the form of springs. In a case where a surface connection is made from the river to the porous material in which wells are driven, the water may percolate through the porous material and reach the wells in this manner, but the writer can recall no considerable success, either from experience in America or abroad, in obtaining large volumes of satisfactory water in this way.

Considering the Parkersburg wells proposed by Messrs. Knowles, Fuller, and Fuertes, the writer would assume that all the supply furnished by these would be obtained from the water moving toward the river from the land side, or pocketed in the porous material along and under the river bed; and though the stages of the river will have an influence on the elevation of this ground-water, it would not imply any flow from the river to the wells.

Several years ago the writer had occasion to test the influence of continuous pumping for 2 months from a gallery or well sunk in a gravel bed along the Iowa River, and, for the purpose of defining the extent of the water field, several observation wells were driven at distances ranging from 100 to 8 325 ft. from the gallery. A level

Mr. J. W. Hill.

base and bench-marks were established, and the levels of the water in the gallery and in the observation wells before and during pumping were noted from time to time and carefully referenced to the base. The pumping was continuous, day and night, for 62 days, and the elevations of the water in the gallery and in the observation wells, at the beginning, during, and at the end of this period are shown on Fig. 15.

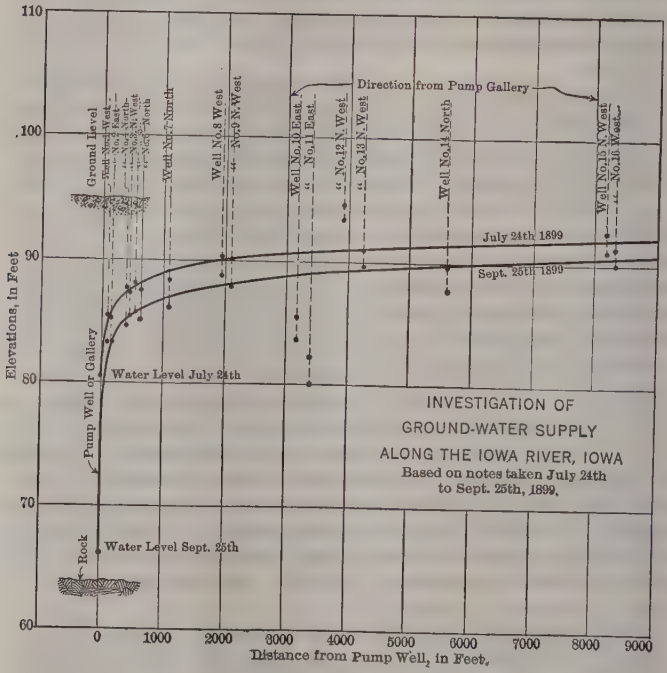


FIG. 15.

Although the test wells shown on Fig. 15 range from 100 to more than 8 300 ft. in a direct line from the pumping well or gallery, they were distributed east, west, and northwest from it, so as to include the water field which it was thought would be affected by the test. Fig. 15 shows some curious results, with reference especially to observation wells Nos. 10, 11, and 12, all of which were open and penetrated the sand and gravel water-bearing strata, and in which the water levels fell from time to time during the test.

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Though the water in No. 12 dropped 1.20 ft. during the 62 days of pumping, at the end of the period the surface of the water therein was considerably above the general elevation of other wells nearest to it in point of distance and in the same general direction from the pump well (northwest).

On the other hand, the water level in Nos. 10 and 11, which were east of the pump well, was at no time as high as in the nearest wells in point of distance, although not lying in the same direction from the pump well. The only explanation for this discrepancy that could be given at the time of the test was that the wells were on lower ground and nearer the river, and might have been supplying the river rather than the test pump well. In attempting to draw smooth curves on the diagram for the elevations at the beginning and end of the test, Nos. 10, 11, and 12 were omitted, all the others lying sufficiently near to a smooth curve to indicate the probable water level at the beginning and end of the test.

"Reference to the diagram, upon which have been plotted the elevations of water in the pump well and observation wells, will show the mean curve of water level which existed in the known area affected by pumping on September 18th. All water in the field above this curve has been pumped away or lost by natural causes, and for the rate of pumpage from the test pump well at that time [693 000 gal. per day] this curve represents the average condition throughout the field.

"The facts obtained from the experimental work [in Iowa] conducted during the summer [1899] are as follows:

"The total amount of water pumped from the test well from July 24th to September 21st, inclusive, was 38 796 662 gal., an average of 646 600 gal. per day. During this interval of time, from the data submitted, the water level in the test pump well was lowered from Elevation 87.78 to Elevation 73.93, or 13.85 ft.

"The total rainfall reported after July 24th aggregates 6.03 in., and, applying this to the area embraced in the investigations, *viz.*, 1 367.4 acres, and allowing an absorption of 40% by the sand and gravel overlying the rock and under the top soil and clay, the absorbed rainfall within the field of observation amounts to 89 555 837 gal., or more than twice the pumpage from the test pump well for the interval of time during which the rainfall was reported.

"The loss of head or lowering of water level from all causes within the field of observation during 57 days of pumping was approximately 2 ft., equivalent to a loss in the field of 235 251 147 gal., to which add the estimated absorbed rainfall, making the total estimated loss from all causes 324 806 984 gal., of which the pumpage from the test well was about 12 per cent."

An interesting part of Mr. Hall's paper relates to the cost of the Smith system of filtration. This is stated as $80 700 for a capacity of 170 000 gal. per hour, or 4 000 000 gal. per 24 hours, or more than $20 000 per million gallons of daily capacity. This is a very high cost

Mr. J. W. Hill.

for a system of filtration of this kind. Elaborate slow sand filters of the type adopted by Philadelphia, Pittsburgh, Washington, etc., cost no more, and are of such a character that the operation may be observed from day to day, and controlled in capacity and efficiency; and rapid mechanical sand filters would have cost at Parkersburg (in 1911) about one-half as much per million gallons of capacity as the Smith system.

The performance of slow and rapid sand filters can be fully controlled, and maintained at high efficiency, regardless of river stages and other unavoidable conditions which are calculated to operate against sand filters of the Smith type.

The guaranty of 4 000 000 gal. per day for a total area of 1.65 acres is equivalent to 2 426 000 gal. per acre per day, or about the capacity of plain sand filters operating without any preliminary filtration.

The manner of making payments for the Parkersburg filter was unusual. A retention of 55% of the contract price for one year after completing the construction indicates that there must have been a sublime faith, on the part of the contractor, in the final performance of the filter system.

Mr. Potter.

ALEXANDER POTTER,* ASSOC. M. AM. SOC. C. E.—The City of Parkersburg is to be commended for its courage in putting in operation the so-called natural slow sand filtration method of procuring a supply of pure water. This method has been advocated so vigorously by its proponents and has been opposed so bitterly by its antagonists that a plant of this kind actually constructed should prove of a distinct advantage in demonstrating what is of scientific value in the plan and what of a distinctively personal bias on both sides of the much controverted subject.

The speaker has watched with much interest the attempts which have been made from time to time to solve successfully the problem of so-called natural filtration. There can be no doubt that the successful establishment of such a plant will meet with popular favor, and therefore such a project should be given a fair and impartial trial.

The speaker recalls that on an invitation for competitive plans for a new water supply from the City of Evansville, Ind., in 1893, fourteen or fifteen engineers submitted plans. In this competition George S. Davison and W. G. Wilkins, Members, Am. Soc. C. E., were awarded the first prize, the speaker received the second prize, and Arthur S. Tuttle, M. Am. Soc. C. E., the third prize. The plan recommended by the speaker was submerged filter galleries on a sand-bar in the bed of the Ohio River, and, as he recalls it, most of the fourteen competitive plans contemplated similar treatment.

* New York City.

Mr. Potter.

Though the speaker's project was not developed by the city, the study made at that time convinced him that, given proper river conditions, infiltration galleries can be designed to give a uniform and pure effluent.

The uniformly good results secured in temperate climates, however, by rapid or slow sand filtration, clearly limit the field of operation of the so-called natural slow sand filtration process, or infiltration galleries, to places where the cost of the latter—the first cost plus operating charges—is less than mechanical or slow sand filtration. In the tropical zone, however, a strong prejudice exists—and rightly so—against the use of stored water, even when filtered; consequently, the development of a supply from infiltration galleries warrants a higher cost than is involved in the construction of mechanical filtration with adequate storage.

Though the analyses of the water included in the paper show a satisfactory effluent, they represent, unfortunately, only a limited period. From the statement made by the author, it appears that radical changes must be introduced in the details of the system in order to prevent a recurrence of the condition found when the beds were examined after having been in service about 3 years. The author states that conical holes were discovered in the sand, having diameters of from 5 to 10 ft. and from 3 to 3½ ft. deep.

It is evident that at all stages of the river, except extreme low water, difficulties will be experienced in maintaining a uniform depth of sand over the filters, and that, under certain conditions beyond the control of the operator, raw water in large quantities may pass directly into the city supply, thus breaking down the barrier against typhoid fever. If no adequate remedy can be provided, either to prevent blow-outs due to back-flushing, or to restore positively and promptly the sand over the strainers when such blow-outs occur, the system particularly described by the author must be considered a failure, no matter how successful it may be most of the time.

With Messrs. Gray, Hall, and others, standing sponsor for the success of the Parkersburg system, it is to be hoped that the best and most intelligent care will be given to its proper maintenance, and that every effort will be made to correct the defects already apparent, which prevent it from producing a uniformly dependable effluent at all times.

A water supply is not built for a few years, or for a decade, but for a lifetime, and because there are those who prefer some other system, no matter what the local conditions may be, engineers should not blind their eyes to the importance of giving a fair trial to this method of water purification.

Passing to the question of infiltration galleries proper, there is no good reason why they cannot be designed to maintain a constant flow

Mr. Potter.

and not gradually decrease and ultimately become obsolete, as is generally the case. They should be as dependable as the identical natural process of ground-water seepage into surface streams. An infiltration gallery, to be permanently successful, must never be forced to such an extent as to allow it to be partly emptied. When it is partly empty, high velocities are set up in its vicinity, which, in time, cause the silting up of the filter media immediately surrounding the gallery, which condition slowly but inevitably results in the breaking down of the system.

It is not a difficult matter to regulate the output from a given infiltration gallery so that it cannot exceed a predetermined maximum discharge, and, by so doing, the lowering of the ground-water plane below the top of the gallery can be prevented, which condition will keep entrance velocities within safe limits and thus maintain indefinitely the integrity of the system.

Throughout the country there exist large deposits of sand and gravel which, if intelligently treated, can be counted on to yield large supplies, and there are also sand and gravel deposits in which the depleted water supply can be augmented by the construction of large storage reservoirs over such a system of infiltration galleries. These reservoirs function as feeders to the infiltration galleries constructed within the confines of the subterranean basin.

The maximum rate of downward filtration which can be maintained permanently is still a mooted question. In the Morris Canal, the leakage, after 100 years of service, averages at the present time 1 250 000 gal. per mile, or from about 200 000 to 250 000 gal. per acre per day. A filter gallery constructed parallel and adjacent to the Morris Canal could be counted on to derive indefinitely 1 250 000 gal. per mile per day from the canal proper. Although feasible to draw off water for short periods at higher rates than this, such an attempt might prove disastrous, as rates of leakage in excess of the natural flow might cause the bottom of the canal to silt up, resulting in a possible break-down in the gallery.

The attention of engineers must be devoted more and more to the determination of safe yields from such underground supplies and galleries, and to the method of controlling the draft therefrom to predetermined and safe quantities.

The speaker is in hearty accord with Messrs. Fuller and Fuertes as to the wisdom of changing the present method of determining and reporting *B. coli*. When the warning signal rings just as loudly at the approach of a cow, or a tramp, walking on the railroad track, as it does for the fast moving express train, one is likely to have contempt for the signal or demand its change. When the presence of *B. coli* may mean evidence of contact with anything, from the human intestine to

Mr. Potter.

the covering of a kernel of wheat, one of two things must necessarily happen: contempt for the warning, or unnecessary and unwarranted alarm. It is difficult to say which is the more damaging.

This unnecessary alarm is exhibiting itself in the laws proposed for enaction in certain of the State legislatures. Certain legislation existing or proposed in many of the States indicates clearly that the authors or the sponsors for such legislation lack the fundamental principles of sanitary science. These laws seek to prevent the entry of sewage, either crude or purified, into the waters of the State. Such a bill is now before the New Jersey Legislature, submitted under the very attractive title, "An Act to Prevent the Pollution of Sources of Potable Water Supply in This State." The principal provision is contained in Paragraph 2, which reads as follows:

"2. It shall be unlawful for any person or persons, private or municipal corporation, to discharge into any water which may constitute or form part of a source of potable water supply any raw or treated sewage, or the effluent from any sewage disposal works, after the date fixed by the State Board of Health when such acts shall cease. Notice to discontinue the depositing of such sewage or effluent shall be given by the State Board of Health to such person, or private or municipal corporation, at least three months before the date fixed by said State Board of Health as aforesaid."

Other provisions of the Act leave the State Board of Health no alternative but to enforce the law prohibiting the discharge of the effluent of any sewage disposal plant, no matter what its degree of purity may be.

Another bill, almost as drastic, has been made a law by the State Legislature of Texas, and, in Oklahoma, the Health Officer, 6 years ago, laid down the principle that neither crude sewage nor purified sewage would be permitted to enter the streams of the State. Even New York State is now seriously contemplating the abandonment of hundreds of thousands of dollars spent on new public institutions, because the authorities feel that they cannot even trust the engineer to design a plant which will insure the conveying of the effluent by pumping from an efficiently designed sewage disposal plant outside of the water-shed of a portion of New York City's water supply.

This idea, which is gaining momentum, carried to its absurd but logical conclusion, means that no city in the United States can discharge its sewage into any body of water which is not tide-water. It is the speaker's opinion that the matter is sufficiently important to demand the serious thought of this Society, and he would suggest that the attention of the Board of Direction be especially directed to this matter, to the end that some intelligent measures may be taken by the Society to counteract this dangerous proposition.

Mr. Fuller

GEORGE W. FULLER,* M. AM. SOC. C. E.—This is quite an unusual paper. It deals with local politics and personalities, exploits some of the general features of the laws of hydraulics pertaining to underground water supplies, and closes with a description of a filtration project which is unique.

In association with James W. Fuertes, M. Am. Soc. C. E., the speaker reported on the situation, after Morris Knowles, M. Am. Soc. C. E., had been called to Parkersburg to advise as to whether a well-water project was feasible, and preferable to filtration of the polluted and impure water of the Ohio River.

Mr. Knowles' general viewpoint was that a well-water project was a fairly feasible one. That report produced some differences of opinion in the community, with the result that a Board of Engineers was appointed to make further investigations.

The speaker agreed with Mr. Fuertes that a mechanical filtration system was to be preferred to a well-water system. Whatever merit there may be in the Fuertes and Fuller report is not in the detailed results set forth in the paper, but rather in the careful application of the fundamental principles involved.

Effort was made to explain, as engineers, to a community that had its well-defined factions, the underlying principles involved. In outline, the fundamental propositions involved are: first, that, for an adequate underground water supply, it is necessary to determine whether or not there are sufficient porous water-bearing strata; second, where does the water come from, which supplies the strata; third, where, under present conditions, does that water go; and, fourth, what is the best way to collect that water for municipal water supply purposes.

Some rather unusual conditions are found above Parkersburg, in that there is a rocky promontory, a few miles above the city, which makes a rock dam across the Ohio River. Consequently, there is no large flow of water in this vicinity beneath the bed of the river. The old river-bed undoubtedly swung to the south, against the West Virginia foot-hills. The surface water from those impervious hills came out above an impervious stratum at the surface of the bottom-lands, and found its way into the river, not through the underground porous strata, but as a surface stream flowing over the impervious clay and silt directly to the river.

Then, it was found necessary to reckon with a partial flow from a very limited drainage area of the river plain, or bottom-lands, plus the underground flow of water to the wells from the adjacent Ohio River. The various hydrographs in the paper show some rather unusual conditions. They show in this river plain a very marked rela-

* New York City.

tion between the widely fluctuating water level of the river and the water level in the deep gravel strata into which the wells were driven. Mr. Fuller.

The obvious deduction was that during high river stages there was plenty of water which would flow to wells that might be sunk in the bottom-lands near the river. When the river level was very low, there was not only a gradually receding water-table in the bottom-lands, with the water from the land side making its exit into the river, but there was also the very striking difficulty of having only small volumes of river water flowing back in a reverse direction from the river to the system of wells.

That small increment of naturally filtered river water in the gravel strata was due, of course, to the blanket of silt and clay which formed on the bottom and sides of the Ohio, at times when the flow down the river was insufficient to produce a scouring velocity to push it away.

The ground-water proposition was abandoned for the reason that there was not water in ample quantities coming from the land side, and, as to that coming from the river side, it was very uncertain whether at times of low water the blanket or carpet of clogging materials on the river-bed could be disturbed so as surely to allow water to reach those wells. Mechanical filters were recommended.

S. M. Gray, M. Am. Soc. C. E., was called in to review this proposition, after an injunction suit precluded carrying out the recommendations of the Fuertes and Fuller report. Mr. Gray recommended a project which, although it may have been adopted in certain respects in other communities, was new in its entirety, as far as the speaker knows, in a city of this size. The Smith system appears to be essentially an old-fashioned slow sand filter layout, without any impervious concrete bottom, without any concrete walls, and with reliance placed on two things to maintain the supply during the very varying stages of the Ohio River, ranging from about 5 to 60 ft., from minimum to maximum, in a cycle of a few months.

During the high stages, the velocity of the flow is so great as to scour away the silt deposits on the surface of the sand bed. To maintain quality and quantity of output at times of low river stages, when there is not a scouring velocity for the removal of surface deposits, Mr. Gray has endorsed, and there has actually been built at Parkersburg, a project with some of the earmarks of a mechanical filter. It has a pipe system scattered over the bottom of $1\frac{2}{3}$ acres of sand resting on gravel of one grade, and, at intervals, it is proposed to back-flush one-fifth of that area, or $\frac{1}{3}$ acre; and on that $\frac{1}{3}$ acre, it is proposed to use about 40 000 gal. during an interval of 15 or 20 min. to remove these clogging materials.

From the standpoint of hydraulics, the speaker is a little disappointed that we have not obtained from Mr. Gray or his assistant, Mr.

Mr. Fuller. Leland, some of the details with respect to that proposition. It brings up several queries: In the first place, as regards cleaning mechanical filters, the fundamentals are, in a rough way, that the summation of exit area of the orifices through which filtered water has to pass should be less than the area of the distributing pipe. Otherwise, the proper pressure is not obtained to force water through every orifice throughout that area, so as to lift and wash the sand thoroughly, say, with a vertical rise of wash-water of from 8 to 12 in. per min. At Parkersburg, the sand is not cleaned so uniformly, but the facts as to distribution of wash-water are not made plain.

Of course, it must be recognized that this is not a proposition directly comparable with a mechanical filter, where wash-water is sent at high velocity through a strainer system under every square foot of filter bed in order to give a thorough cleaning. This Smith system seems to be one of those arrangements where back-flushing is used simply to lift the clogging material sufficiently so that whatever flow there may be in the river may be sufficient to move that material beyond this area.

Slow sand filters are back-filled, of course, very slowly, in order to remove air, and Kirkwood's book on filtration states that, more than 50 years ago, such a filter at Tours, France, was cleaned by back-flushing. However, it is stated by Kirkwood* that the head of water was insufficient and the process was not successful.

To build this filtration system of 1⅔ acres at Parkersburg has cost $80 000. That is a somewhat larger sum than was spent at Lawrence, Mass., for building in 1893 an open filtration plant extending over 2½ acres. The speaker does not recall what Mr. Fuertes and he reported with respect to the cost of mechanical filters at Parkersburg, but is inclined to think that it was markedly less than that of the Smith system.

Speaking about costs, mention should be made of one point that is set forth on page 752 of the paper, where it is stated:

"It is gratifying to note that the new pumping equipment has effected an economy sufficient to pay the capital charges on the bond issue required by the new plant."

As a citizen of Parkersburg and a member of the Water Board, Mr. Hall is entitled to a great deal of commendation if he is able to establish a new and reliable plant without any added cost to the tax-payers; and the low operating expense of the Smith system is certainly one of its strongest points.

This Smith system, which is understood to be patented, has not been exploited at many other places. At Henderson, Ky., it was reported on by Philip Burgess, M. Am. Soc. C. E., and found to have

* Pp. 125–126.

some local merit. George A. Johnson, M. Am. Soc. C. E., and his associates at Wheeling, W. Va., reported on it at great length, with unfavorable conclusions. Mr. Fuller.

It is interesting that, within the past 6 months, Mr. Leland has overhauled the sand and gravel, in connection with back-flushing, rather early in the period of drought in 1916.

With regard to quality, the water furnished by the Smith system is generally of good appearance. In point of hardness, it does not vary markedly from that of the Ohio River. In that respect, it compares with what would be produced by a filter plant of any kind.

With regard to the bacteriological aspect of the water, it is doubtful whether it is of as good a quality as would be afforded by any of the so-called standard modern filter systems or as good a water as the people of Parkersburg ought to obtain. It is rather doubtful whether this water supply is able to comply with the standards of the United States Government, as set forth by the United States Public Health Service, a branch of the Treasury Department, *viz.*, not more than 2 *B. coli* per 100 cu. cm.

That Department has provided certain tests for intestinal bacteria in waters supplied to Interstate carriers. Looking over the analyses on page 786, it is doubtful whether the City of Parkersburg could sell water to the railroads which pass through it and not get into trouble, if there should be a thorough analytical study of the water.

In making that comment, it is only fair to state that this is a wonderfully transitional age, in point of water quality. Many cities are supplying water of the highest type in point of quality, either by filtration or by the elimination of sources of pollution. By this means the typhoid death rates of American cities have been reduced to a point which seemed almost inconceivable 10 or even 5 years ago. We have death rates to-day, in cities supplied with filtered surface water, which are as low as those found in European cities, in Holland, and in certain Rhenish districts, where they have pure water from underground sources.

There is, however, a very anomalous situation prevailing in America to-day, in that in some of the surface water supplies there seem to be bacteria of many different kinds, which are classed by laboratory men as members of the *B. coli* group. Some of them are said to be the sour milk bacteria—whatever that may be. Others are said to be bacteria which grow on grain and are found in soils.

In certain districts, where there is a purely rural population, with here and there a scattered village, it is said that there are water supplies which are so polluted, according to laboratory evidence, that they could not reasonably meet the Government standards, even after filtration.

Mr. Fuller.

In relation to this matter, it may be stated that there will be published within the next few weeks a new edition of the so-called "Standard Methods of Water Analysis" (American Public Health Association), the purpose of which is to separate some of these forms of bacteria from the so-called *Bacillus coli*. Just where it will lead is not known, but the situation to-day, in point of quality of water supply for American cities, is a very anomalous one. It is not readily known whether any given city is living up to the legal requirements of the Federal Government; and, if it is not, it is not known whether the apparent fault is genuinely attributable to sewage pollution, whether it is due to laboratory caprices in reference to the sour milk bacteria, whether it is due to harmless bacteria that grow on grain, or whether it is the fault of the handling of water by filtration, or otherwise.

It is hoped that the next year or so will see a sharp turn in the road, a change from the uncertainty of the past to a definiteness of assignment between cause and effect, so that there surely will continue the improvement in quality which is one of the striking features of the past generation in the point of municipal water supply.

The speaker is glad that the author has presented this paper. It sets forth some simple truths which could be considered to advantage by all who are interested in the subject. He has not been very prolific in setting out the specific details of the Smith system, and it is to be hoped that more data may be obtained later.

Mr. Potter's remarks bring up a question that is very active in the interpretation of the treaty passed in 1912 between the United States and Great Britain, with respect to controlling the pollution of boundary waters. There has been under discussion now for about 3 or 4 years the wording of a procedure to stipulate plainly what shall be the quality of the boundary waters, so that life or property on one side of the boundary shall not be affected prejudicially by those who discharge sewage into these waters on the other side. After the many efforts made by the Canadian and United States Governments to solve that question, it would be better to wait until we get further evidence from those who are at present studying it.

With regard to Mr. Potter's remarks on the so-called intestinal bacteria, some men want to exploit water analyses very quickly in laboratories, where the results are to be used by the men responsible for the management of a filter plant, or by those who direct the endeavors of a large number of inspectors on a water-shed. Presumptive tests are not well thought of by those who desire to form an opinion on the intrinsic merit of the sanitary quality of water, and especially in the comparison of different waters. For the past 10 or 20 years there have been growing up two factions with respect to

applying laboratory methods to the control of water supplies. The men in one set want to get results in 24 hours. They have had their way for the last 5 or 10 years. They have brought into municipal water reports, during the last few years, a set of results which have helped the filter operator, but, to the men who have given years in attempting to control water pollution, these results are open to grave suspicion and are exceedingly difficult, if not impossible, of interpretation. Mr. Fuller.

On considering these figures in type in a municipal report, without any personal knowledge of the environment to which these data apply, trouble ensues. Some of these bacteria, classed as *B. coli,* innocent as they may be, may multiply and breed in water under certain temperatures. This certainly adds to the complications. There ought to be a marked change with respect to laboratory data for the sanitary control of the quality of water supplies in America. Engineers must recognize two distinct phases, one, the quick result to help the filter operators and the men who direct inspectors. When it is desired to get a measure of sanitary quality, however, it is necessary to go deeper and further, and to take 8 or 9 days for analysis, something which the filter operator will not tolerate. On the proposition of telling whether or not a water will measure up to prescribed sanitary requirements, it seems highly important to make every effort to separate fecal from non-fecal bacteria in the laboratory.

Nicholas S. Hill, Jr.,* M. Am. Soc. C. E.—Mr. John W. Hill and Mr. Fuller have covered this paper so thoroughly that it leaves very little to be said in the way of discussion. The speaker certainly agrees with both of them in their criticism of the Parkersburg system. Mr. N. S. Hill, Jr.

The situation in Parkersburg recalls a condition that so often exists, and the futility of expert reports under certain conditions, that is, the natural tendency of people to adopt a so-called pure underground water supply, with all its uncertainties as to quantity, rather than the acceptance of a surface water supply of known abundance, treated with modern processes which have a known efficiency.

The use of underground waters is not without dangers; and, without discussing this paper in detail, it might be well to call attention to a situation which sometimes arises where underground water supplies are developed. The speaker alludes more particularly to the East Orange water supply in New Jersey, and will refer to the case of Harper *versus* the Mountain Water Company. This case was tried at Somerville, N. J., before the Hon. William J. Magie, then Chief Justice.† The following is a passage from the Judge's charge:

* New York City.

† A convenient summary of this case, by Vice-Chancellor Emery, may be found in 20 Dickinson, p. 479.

Mr. N. S. Hill, Jr.

"I think I should first give you what the law defines as the rights of parties respecting water running in streams. Any one owning on the banks of a stream of running water is entitled by law to have the waters of the stream come down to him in their natural flow, uncontaminated, unreduced, and unobstructed, except so far as Nature affects them. Persons owning above the line of the person who has these rights have also rights in the water passing them, and their right is to have the water pass them and go down to those below unaffected by obstruction from below or unaffected by obstruction or abstraction from above; each of these parties has a right to the use of the water, which is a reasonable use of the water as it passes. Water may be thus used for irrigation, for the watering of cattle, for the use of water around buildings and houses, for the use of water to produce power and thereby run mills or whatever else power may be applied to. I say the riparian owner, that is, the owner on the bank of the stream, has the right, the right to water coming down from above unobstructed, unpolluted, and in its natural flow."

The same rule has been applied to a clear case of a subterranean stream, and apparently no distinction is drawn, whether the underground water is regarded as percolating water or water of a well-defined subterranean stream.

Vice-Chancellor Magie proceeded to say that though a person or company might strike underground streams and abstract underground water which "might" come to the surface, and might carry this percolating water off and sell it, yet, if by their works they abstracted the water or reduced the level of springs or streams which "had" come to the surface, it was an actionable wrong. The words "might" and "had" in this case are emphatic. The jury was directed that, if it was shown by a preponderance of evidence that the defendant had abstracted an appreciable quantity of water which "had" come to the surface in a pond, spring, or stream, so as to diminish the flow, and had done this either by pumping from the stream or from a well, or by both, then the plaintiffs were entitled to damages.

Later, there has been the decision in the case of Frank Meeker *versus* the Mayor and City Council of East Orange. The City of East Orange recently established an underground water supply, deriving its water through wells about 150 ft. deep. The plaintiff stated that, before and at the time of the establishment of wells, he owned certain property and lands in Millburn and Livingston, having an area of about 100 acres, that by reason thereof he was entitled to the benefit and advantage of a certain spring and stream running and flowing into and through those lands, and that the defendants, by Artesian wells, pumps, and reservoirs, did, between January 19th, 1905, and September 25th, 1906, divert large quantities of water from said springs and streams, to the prejudice of the plaintiff.

Mr. N. S. Hill, Jr.

Judge Adams, before whom the case was tried, charged the jury as follows:

"If you are satisfied that if, by the operation of the defendants' works in intercepting underground waters, any existing natural surface stream was diminished in its flow on the land of Mr. Meeker to an appreciable extent, he has suffered an actionable wrong, and is entitled to compensation."

Furthermore, he stated that the alleged diminution of the surface streams and the alleged destruction of the run-off from the springs are claims independent of one another, but covered by the same legal principle. He referred to the Harper decision, and stated:

"It is not easy to determine just what inferences are legitimate from the language used by His Honor, Chief Justice Magie, in dealing with a case similar to this and yet not altogether analogous. On the whole, I conclude that the Harper case warrants this Court in saying that when an existing spring, actual, not merely potential; visible, not occult; a spring, not a well; brought to the surface by natural force, not conducted there by the labor of man—when such a spring is depleted by subterranean suction occasioned by the operation of a pumping plant like that at East Orange, an actionable injury is inflicted upon the owner of the land on which the spring is situated, for which the law will award sufficient compensation."

In another case, Smith *versus* the City of Brooklyn (160 New York, page 357), the following principle was laid down. If a city, by the operation of a water system, consisting of wells and pumps on its own land, drains the contiguous territory, and thus diverts and diminishes the flow of water in a natural surface stream on the land of another, it is answerable in damages, under the law that no one may divert or obstruct the natural flow of a stream for his own benefit to the injury of another.

Under these decisions, no provision whatever has been made for the assessment of permanent damages. A man brings an action for the damages resulting from the diversion of underground waters for a definite period, and asks to be reimbursed for the damage which he has suffered during that period. With surface streams the damages awarded are permanent, and the Courts have defined their measure as the difference in the value of the property before and after diversion. The result of these decisions has been that East Orange has been in constant litigation since 1905, when its underground system of water supply was started, and the City has been compelled to acquire by condemnation more than 1 750 acres of land for which it has no specific use, in order to prevent continuing damages from accruing from year to year. This property has cost the city $400 000 more than the original estimate for the water supply system, or approximately 25% of its value.

Mr. N. S. Hill, Jr.

There are many situations in which such a condition may develop, and every engineer should use great caution in advising a municipality to embark in an underground water supply before satisfying himself that such a condition is not likely to arise.

The speaker simply calls attention to this added element of uncertainty, in conjunction with consideration of this paper. It is not always possible to develop underground water supplies and be entirely free from claims for damages to surrounding property, as many suppose.

Mr. Spear.

WALTER E. SPEAR,* M. AM. SOC. C. E.—It would be pertinent to the discussion of this paper to speak of two of the larger German ground-water works which derive at times a large portion of their supply from adjacent streams in somewhat the same manner as the Parkersburg works. The collecting works of the first of the two large ground-water plants supplying Dresden—the Saloppe Works—with which many members of the Society are doubtless familiar, consist of a slotted cast-iron pipe, about 4 700 ft. long, and from 18 to 26 in. in diameter, which is laid along a low bank of the river about 50 ft. from the edge of the water at ordinary low stages in the Elbe, and about 8 ft. below the river surface. The cover of sand and gravel over the gallery is about 15 ft. The maximum yield of this water in 1904 was given as 12 000 000 gal. per day. Originally, it was believed that all the water obtained at this plant represented the ground-water flow from the upland water-shed that came slowly through the gravel strata in the broad river valley. This idea, however, was soon abandoned. At high river stages, when the ground over the gallery was flooded, it was found that very considerable quantities of water came directly from the river through the 15 ft. of cover. Ordinarily, the water from this gallery was of very good quality, but, when the river was in flood, and a large proportion of the supply was filtered river water, it showed some deterioration. During a flood in 1896, when the river was 10 ft. above the surface of the ground over the gallery, there were several thousand bacteria per cubic centimeter in the water delivered at the station. There was no typhoid fever in Dresden as a result of the flood, but many cases of intestinal disorders were reported in the city.

The Schierstein Works, supplying Wiesbaden, at the time of the speaker's visit in 1904, drew a supply of water from 32 wells driven along the Rhine at a distance of from 200 to 400 ft. from the water at low river stages. These wells were about 35 ft. deep, and penetrated a thick stratum of sand and gravel, which was covered for the most part with a layer of sandy clay 10 ft. or more in thickness. Ordinarily, the supply from these wells was of good quality. It was practically all ground-water, but the cover of sandy clay was known to be absent

* New York City.

in the river channels, and the director of the works, Mr. Halbertsma, considered it wise to put in an ozone plant at the works to sterilize the water when the Rhine was in flood. Mr. Spear.

In looking over the description of the Parkersburg collecting works and noting the meager thickness of cover over the screen sections, it would seem to be the part of prudence for those responsible for their operation to put in a sterilization plant. Although such a plant might well be operated all the time, it should certainly be used when the river is in flood, when the surface of the filter, therefore, is inaccessible, and when the analysis of the water shows signs of pollution.

The Parkersburg plant has some slight resemblance to artificial ground-water works constructed extensively in Sweden, where there are natural beds of sand and gravel, as at Parkersburg, without any adequate supply of ground-water. At these works in Sweden surface water is applied through open channels to the surface of these beds of pervious material, and the water filtering through is abstracted below, generally from wells driven at a safe distance from the open channels. The surfaces of these beds may be cleaned like a filter, by cutting off the supply and allowing the water to drain away. In respect to the depth of material through which the water flows to the wells, and the readiness with which the filter surface may be cleaned, these Swedish works differ from the Parkersburg plant, and these points of difference seem to constitute the chief elements of weakness in that plant.

T. Kennard Thomson,* M. Am. Soc. C. E.—This paper is of special interest to the speaker because he has long had a high opinion of the author, a graduate of West Point, and has himself had considerable experience in, on, and with the Ohio River, first at Kenova, about 125 miles south of Parkersburg, where the depth was 6 ft. at low water and 66 ft. at high water, with very sudden fluctuations or floods; then at Mingo Junction, about 75 miles above Parkersburg. Mr. Thomson.

The speaker also designed caissons for the Wabash Bridge over the Monongahela River at Pittsburgh, just above the junction of the Monongahela and Alleghany, about 200 miles above Parkersburg.

Mr. Fuller has mentioned the impurities of the Ohio River water, and his reference can readily be understood by any one who knows the river. In the first place, to give some faint idea of the sediment carried by this river, it might be stated that, during the winter of 1890-91, one of the coffer-dams for the Kenova Bridge, which was approximately 30 by 100 ft. and 25 ft. in depth, was left in the river. When the waters subsided in the spring this coffer-dam was found to be filled to the brim with sediment. The speaker has no means of knowing whether or not the river could have filled it more than once during the winter.

* New York City.

Mr. Thomson.

A river which carries such an immense quantity of sediment and is subject to such severe floods and dry spells is not very dependable, to say the least. To make matters worse, the river is not only polluted by the discharge from many sewers, but, also, during these periodical floods, by the water washing and receding from the streets, cellars, houses, stables, barnyards, chicken coops, etc., of many towns and cities, Pittsburgh included. In fact, one would imagine that the water would be much better for fertilizing crops than as a domestic supply.

The speaker thinks the Parkersburg authorities were wise not to try to pump 3 000 000 or 4 000 000 gal. of water a day from fourteen wells placed some distance from the shore; for he does not think it possible to pump such a volume of water continuously without removing more or less sand, perhaps in almost imperceptible quantities, but still sufficient to form cavities.

These cavities would probably cause no damage in the open fields, but, sooner or later, these fields will be built on by the city; and though the subterranean cavities might even then give no trouble for many years, they might sometime, suddenly, cause very serious collapses, similar to those which have happened to cities built over coal mines. The experience gained by Parkersburg through its adopted plan would seem to prove the correctness of this assertion.

Some day all the towns along the Ohio (and this applies also to other rivers) will combine to dam the water up in the mountains, where it is comparatively pure and free from sediment, and pipe it to the different towns as needed. This will not only give an adequate and dependable supply of good water, but will also regulate the flow in the rivers, doing away with the disastrous floods of periodical occurrence. As an example of many towns along the Ohio: when the speaker first saw Catletsburg, Ky., the town stood on a bluff some 20 ft. above the river. The next spring he saw rowboats landing at the second-story windows of the houses. These floods occur so quickly that, even with the excellent modern Weather Bureau service, the inhabitants do not always have time to protect their property; yet they come back, clean their houses, make the best of their losses, and start over again; they very seldom move away or build on high ground.

The Court decisions quoted by Mr. Hill would seem to indicate that the small towns may some day be able to force the towns above them to provide pure water or to stop polluting the rivers.

Mr. Dunham.

H. F. Dunham,* M. Am. Soc. C. E. (by letter).—A paragraph on page 780 giving the dimensions of the intake pipe could be illuminated by mention of the connections to, and the character of, the pumping machinery. It would appear from the text that the large diameters of the intake or "suction pipe" were expected to lessen the otherwise

* New York City.

"undesirably rapid rate" of flow through the filtering material. This implies the use of intake pipes as reservoirs from which a certain quantity of water could be taken without marked or legitimate effect at the strainers. Is this view correct? Ordinary experience would tend to show quite the reverse; that the friction losses would be less and the draft at the wells greater with large pipes under a vacuum produced by pumps of any description. Mr. Dunham.

Strainers with V-shaped openings have been used by the writer, not always in vertical positions, but always at some angle and with connections that would allow of access to each well or to each one in a small group when shut off from the others in a system. Any other method seems to be open to distrust, although it is true that conditions vary greatly, and each case is a problem by itself.

Many years ago, the writer examined sand bars in the Ohio River in the vicinity of Wheeling, W. Va., for the Wheeling Water Board, but found little encouragement in quality or volume, although the beds were continuous to a depth of 40 ft. below low-water levels. Besides hardness, the figures for iron were too high, and filters were recommended.

THEODORE S. JOHNSON,* ASSOC. M. AM. SOC. C. E. (by letter).—In reviewing the problem of water supply, the same questions that arise at the beginning of the study are still to be considered after some one solution is adopted; namely, is the water obtained of satisfactory quality from chemical, physical, and bacteriological standpoints, and, is the requisite quality obtainable at a fair cost of production? Usually, these questions can be answered after some investigation. It is to be regretted that rarely are two solutions of a water supply problem provided so that they may be compared in actual performance under the same conditions of demand and production and at the same time. In this discussion the operation of one type of supply is presented, and an attempt is made to compare it in results with one not in operation. This is a handicap in solving the problem as to the scheme which has not been adopted. Mr. Johnson.

This paper recites the preliminary investigation for a well supply for Parkersburg, and then describes the construction and operation of a system not capable of such preliminary investigation, concerning which little or no previous research or study had been made. It would have been desirable to have answered in the paper the following questions:

1.—How does the present supply compare, in turbidity, total hardness, iron and bacterial content, in color, and in other factors affecting the desirability of a water supply, with the same factors and qualities of the water taken from the well supply as investigated

* Granville, Ohio.

Mr. Johnson.

by Mr. Knowles and by Messrs. Fuertes and Fuller, which supply was rejected?

2.—How does the cost of pumping from the present beds compare with the estimated cost of pumping from the line of wells? Speaking more generally, how does the cost of operation, including capital charges, compare with the same cost as estimated for the rejected supply?

3.—What may be the probable life of the present plant, as compared with that of the plant for well pumping? The object of this question would be to show the effect of depreciation and maintenance charges on the cost of pumping.

The data in the paper are not sufficient to enable the reader to determine the answer to the first of these questions. The reports published in the paper, showing turbidity, alkalinity, and total hardness, are given only for one month, and therefore cannot be regarded as showing very fully the characteristics of the new supply. The figures in the table on page 785 are interesting. For example, the iron content of the raw river water varies from 1.06 parts per million, on January 20th, to 7.04 parts per million on January 24th, 1914. This is a very wide range for iron content in the raw river water in such a short period. Again, the average iron content for the month is 3.33 parts per million. Having been familiar with the untreated Ohio River water from his earliest remembrance, and being at present a user of water in which the iron content is slightly more than 1 part per million, the writer is surprised to find the Ohio River water so contaminated with iron. He is still more surprised to find, on page 783, a report of an investigation of the supply by Mr. I. L. Birner, which states: "The total iron in solution, of the filtered water, was 0.19 of one part per million, against an average of 5 to 16 parts in the river." From the wording of Mr. Birner's report, one is led to believe that he refers to parts per million, and this would seem to indicate a river water extremely polluted with iron. The only data which the writer has immediately at hand are those given in the Report of the Ohio State Board of Health for 1908, on page 206, where results are given for tests made at Marietta, Ohio, a few miles up stream from the site of the present filtration works. These tests show an average iron content for 1906 of 0.62 part per million, and the maxima for the year are found to be 1.00 and 1.23 parts per million for August and November, respectively. It is difficult to reconcile these two statements.

In the same report the bacterial results show the untreated water at Parkersburg to have from 100 to 3 100 bacteria per cu. cm., and the filtered water from 24 to 1, with averages for the month of 900 and 9.3 for the untreated and filtered waters, respectively. Tests

Mr. Johnson.

by the State Board of Health at Marietta show that the average of a large number of determinations indicates a bacterial count of 8 200 per cu. cm. in the raw water during 1906, and 7 350 during 1907. The same investigators, in tests made on July 24th, 1906, at Pomeroy, Ohio, show 19 100 bacteria per cu. cm. in the raw river water and 8 200 on November 19th, 1906. Although these are only scattered results of tests, they serve to cast some comparative light on the published figures. That the safe condition of the water, as indicated by the tests of Dr. Rose and Mr. Birner, is not generally recognized, is shown by the suggestion that a report prepared by the City of Wheeling and tests by the State Bacteriologist of West Virginia both agree that at times the water is quite unsafe.

Engineers may then reasonably inquire whether experience with other well supplies, or tests on the wells as driven at Parkersburg, would not indicate that the quality of the water from the well supply is better than that furnished by the present infiltration system. If there are published or available reports on the well supply, or additional tests on the present supply, which will throw more light on this question, it would seem highly desirable to have them published.

To answer the second question—how does the cost of pumping from the present beds compare with the estimated cost of pumping from the line of wells?—the necessary information is not available in the paper and, by all means, should be presented by the author or by the engineers concerned. The figures on page 781, showing the cost to be $11.03 per million gallons, are very evidently only for the cost of operation, and include no capital or depreciation charges. The data on pages 780 and 781 do not separate items in construction cost in such a way that direct comparisons can be made.

The service life of the present plant is necessarily one of doubt. It is not the time in which the present construction will furnish the proper capacity, with which we are concerned, but rather the life of any one filter unit. Relative to this it is noted that an examination of the beds in 1916 showed that, at several places, there were holes in the sand and gravel covering. These holes were from 5 to 10 ft. in diameter, and were formed like craters, extending to within 1 or 2 ft. of the brass strainer pipes. The writer has been long familiar with the locality in which the filter beds have been placed, and was engaged on some surveying work, in connection with the quantities of excavation under the contract, at a time very near that at which the photograph shown in Fig. 13 was taken. At that time, he was able to walk to the coffer across the slightly raised level of the bar, and, on every side, were to be seen similar crater-like holes in the natural surface of the sand-bar. In some of these there were pools of water in which children were bathing. It has been the writer's

Mr. Johnson.

opinion that if these craters would form in the natural bed, they would be as likely to occur over the filter bed, and, if so, would of course greatly impair its efficiency. Though these holes may be refilled, it would seem hardly possible to conduct the refilling so as to secure uniform stratification in the bed, and the reasonable life of the infiltration plant would be considerably shortened by this cause alone. The efficient life of a rapid sand filter or a slow sand filter is a question which can be answered with fair accuracy. It would not seem that the conditions providing stability in the plant were to be found in the shifting variable sand bottom of the Ohio River.

In a general way, the comparison of this plant with either a filtered water supply from the Ohio River or a supply drawn from wells in the gravel beds of the adjacent bottom-lands can be put on a satisfactory basis only when the Smith infiltration system is regarded as being of the caisson type or as a type of sand filter. If of the latter type, it would seem that the same conditions which have been regarded as necessary to secure a satisfactory supply from a rapid sand or slow sand filter would have to be applied to the Smith system. Among the first requisites for either of the usual filter types is a careful control of the operating head, as affecting the rate of filtration, and both types are considered as sensitive to small changes in the effective heads. With a range of 53.1 ft. in the stage of the river, it would hardly seem probable that the filtering conditions remain the same.

Another requisite of proper sand filtration is a careful control of the cleaning processes. The filters are constantly inspected and watched, and the conditions carefully maintained for securing equal fitnesses and thorough washing of all their parts. If the beds of the Smith system are pitted with crater-like holes extending to within a short distance of the strainer pipes, the efficiency of the back-flushing would certainly be reduced. If, as a matter of experience, such back-flushing does cleanse all parts of the filter bed very thoroughly and uniformly, the data which prove this fact would be interesting.

With a supply of extremely varying nature, such as that found in the Ohio River, sand filtration plants are accustomed to have constant and rigorous tests at least daily, and often more frequently, on the basis of which the rate and character of their operation is altered. Although the author recommends systematic tests, these tests will be lessened in value by the fact that little or no change could be made in the operating conditions of the filter system. It may then be concluded, in the writer's opinion, that the Smith infiltration system may not be regarded as a purification plant, capable of providing at all times a safe and wholesome supply of water such as is obtainable from rapid sand filtration plants, and, therefore, must be considered

as an underground caisson or ground-water collection system, of rather high construction cost, drawing water either from the river above or the ground-water below, and confining its purification processes to mere straining. Mr. Johnson.

PHILIP BURGESS,* M. AM. SOC. C. E. (by letter).—The writer has read this paper with much interest. As stated in the paper, it has been the author's purpose principally to discuss the investigations made by the engineers who were retained by the City to determine the best method of obtaining a satisfactory water supply. The writer was retained by Mr. Smith, the contractor for the system which was finally built, principally to conduct the preliminary investigations which Mr. Smith had made before the construction of the plant, and, finally, to prepare the plans from which the system was actually constructed. The following discussion is offered principally for the purpose of describing the preliminary tests and reports made prior to actual construction. Mr. Burgess.

From his position as an outsider, the writer was particularly interested in the opinions of the comparatively large number of engineers obtained before the City authorities finally agreed as to the system which was considered best adapted to serve the needs of the city. The writer was led to believe that perhaps this condition was attributable largely to the fact that the reports of the engineers as submitted were not sufficiently conclusive and convincing, and that they contained serious differences of opinion.

The first report submitted by Messrs. Chapin and Knowles recommended "that the ultimate object to be looked forward to is the establishing of new water-works at the Camden Farm, with the building of either wells or a mechanical filter plant; the former, if investigations show the feasibility, and the latter, if not so found."

Again, on page 16, of that report, it is stated: "It cannot be too strongly urged that, while the well proposition is attractive, further investigations should be made, perhaps in other localities nearer the proposed reservoir site."

It is apparent that, although this report seemed to favor the construction of a well system, it showed that the engineers were not in a position to make a definite recommendation for building such a system without further investigations.

Under date of February 21st, 1910, Messrs. James H. Fuertes and George W. Fuller submitted to the city officials a joint report, in which appears the following statement:

"We find, as will be seen from the estimates of cost of construction and operation, that a first-class well system and a first-class filter system at the Camden Farm are substantially on a parity as to the quality

* Columbus, Ohio.

Mr. Burgess.

of the water, and as to cost, both of construction and of operation (when a suitable charge for the required land is made against the well system), and that either plant could be depended upon to supply such quantity of water as would be needed by the city for some years. * * *."

The conclusions and recommendations of Messrs. Fuertes and Fuller were very definite, but an examination of their report indicates that these conclusions and recommendations were based on certain assumptions, as shown on page 13 of that report, as follows:

"During wet weather, perhaps when the water in the Ohio River is high, it appears to us that the source of water contained in the pores of the extensive sand and gravel layers is largely river water, but to some extent water from upland sources, the high river stage damming up some of the upland water.

"We are firmly of the opinion that, during low-water stages in the Ohio River, the water which can be drawn from the strata is that which has been stored within the pores of the material at times of previous high water in the river. As this stored water is removed, or naturally flows away, we are certain that, during low stages in the river, whatever water enters these sub-surface strata of sand and gravel is largely of river origin.

"Our conclusion, therefore, is that, as a source of supply for a municipality, the water from the underground sources in question, at the end of protracted droughts, must come from the Ohio River through the bottom or sides of the present river bed along the stretch of the bottom lands below Briscoe."

The conclusions of the engineers as to the source of the gravel-water modified their report, as will be shown in the following discussion. On pages 14 and 15 of the report appear the following statements:

"In the first place, it was found that the water in the porous sand and gravel layers in question did not rise to such high levels as found in the river itself. This is true as indicated by readings in test wells almost at the bank of the river; and shows that, even during fairly high velocities in the river, there is still so much mud and silt deposited on the sides and bottom of the river that a considerable head is required to overcome the friction sufficiently to allow the water, in substantial quantities, to pass through this thin and more or less impervious deposit of mud."

This statement is of interest, in view of the fact that the bottom of the Ohio at Parkersburg is always clean and free from deposits of silt.

In their concluding discussion of the quality of the supplies, the engineers stated as follows:

"There are some slight differences in quality of the water from wells and from filters, the principal feature of which would be hardness on account of the wells yielding water coming largely from the Ohio

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River at times when it is in flood and fairly soft. We do not attach much importance to this difference because ultimately during low-water periods the well water would also have its source in the Ohio River. So far as quality is concerned, we put waters from filters and from wells in the vicinity of the Camden Farm on a parity. * * *"

In describing the well system and its operation, the engineers recommended the following method of increasing the supply during low-water stages:

"*River Plant.* To insure the removal of mud and silt from the bottom of the river adjacent to the well plant at the latter end of low-water periods, we have considered it prudent to provide for a hydraulic dredge located upon a barge equipped with 25-horse power engines, steam-driven, and which would operate not only the centrifugal pumps for dredging purposes, but also some arrangement for propelling the boat at intervals up and down this distance of 4 000 ft. adjacent to the wells. The suction of these dredges would be specially designed, resting upon a shoe and with the end attached to a special casting, with a view to removing say for a depth of 1 or 2 in. the material over as wide a strip as possible. It is probable that such a general device could be arranged so as to clean the bed of the river for a width of 20 or 25 ft. and for a length of 4 000 ft. in a working day. This would give an available filtering area of roughly two acres. * * *.

"We estimate that the annual cost of operation and maintenance of a 4 000 000-gal. well water project, exclusive of fixed charges, would be as follows:

Fuel, based on gas	$11 680
Pay-roll	4 000
Supplies, repairs, and renewals (average)	4 000
River dredging	3 680
Total	$23 360

"We estimate that the annual cost of operating and maintaining a 4 000 000-gal. filter plant in a first-class manner, and of pumping the water to the distributing reservoir, exclusive of fixed charges, would be as follows:

Fuel, based on gas	$11 000
Pay-roll	6 500
Coagulants	2 500
Supplies, repairs, and renewals (average)	4 000
Total	$24 000"

From these citations, it is apparent that the conclusions of the engineers as to the cost of operation and as to the comparative merits of supplies derived from a filter plant or from a well system were largely influenced by their assumption as to the source of the well water. It was this assumption in regard to the source of the well water and in regard to the method of maintaining and increasing the available well water supply during dry-weather conditions, which was

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not convincing or conclusive to, at least, some of the city officials and members of the Water Commission. If one omits the annual cost of river dredging, estimated at $3 680, from that of operating the well system, it is materially less than that of operating mechanical filters, as determined by the engineers. In other words, omitting such cost of river dredging, and assuming the filtered river supply and well water to be equally satisfactory as to quality, the engineers' conclusions and recommendations for establishing a mechanical filter plant were not warranted.

Messrs. Fuertes and Fuller did not agree with the conclusions of Mr. Clapp, who thought that the ground-water at the wells had its source partly from the sand plain districts and partly from the bottom-lands in the main valley of the Ohio.

As stated by the author, the City finally employed Mr. Samuel M. Gray, who submitted a final report, under date of May 26th, 1910, which was so convincing and conclusive, especially as to the merits of the system advocated by Mr. Smith, that the City soon entered into an agreement with Mr. Smith to build his system.

The author has given a very interesting discussion of the preliminary investigations made at Parkersburg to determine the comparative merits of supplies, obtained either by filtering the Ohio River water or by puting in wells adjacent to the river. However, he does not mention any of the preliminary studies made to determine the feasibility of the Smith system. This feature of the work embraced perhaps a more thorough investigation than was made to determine the merits of the well system. Moreover, it is the writer's belief that the final success of the Smith system at Parkersburg is attributable largely to the thoroughness with which the preliminary tests were conducted to determine, first, the feasibility of putting in such a system, and, finally, to determine the best location for the plant.

Briefly stated, the method of conducting the preliminary tests included the sampling of the underflow of the river, as obtained by driving small well points to depths ranging from 3½ to 5 ft. beneath the surface of the bar. Water was pumped from these well points for such a time as was required to obtain a clear sample which was finally analyzed for the purpose of determining its chemical constituents. It was recognized that two important features must be considered in selecting the location for the system, namely, that the underground water must not contain an excessive quantity of iron, and that it must not be excessively hard, as compared with the river water.

The first preliminary tests were made at Parkersburg on August 6th and 7th, 1909. At this time only four tests were made, at stations extending from about 5 000 ft. to 1 000 ft. above the city pumping station. The results of the analyses of the samples collected at this time are shown in Table 1.

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TABLE 1.—RESULTS OF TESTS OF WATER OBTAINED FROM BENEATH THE BOTTOM OF THE OHIO RIVER, BY DRIVING WELL POINTS, AT PARKERSBURG, W. VA.

(Results in parts per million.)

Station No.	1	1	2	2	2
Depth of sample below surface of bar	Surface of river	2.5 ft.	Surface of river	3.0 ft.	4.0 ft.
Sample No.	1	2	3	4	5
Date of collection	8/6	8/6	8/7	8/7	8/7
Time	3 P. M.	4.30 P. M.	8.30 A. M.	8.45 A. M.	9.30 A. M.
Temperature, in degrees, centigrade	28.0	23.0	26.0	28.0	23.0
Turbidity	11	5	17	0	0
Free CO_2	0	23	0	11	18
Dissolved oxygen	9.0	2.0	7.7	....	1.8
Chlorine	....	....	31	....	14
Iron (Fe)	....	5.0	0.5	....	6.0
Alkalinity	15	174	17	57	58
Incrustants	....	....	100	....	20
Total hardness	....	....	117	....	78

Station No.	3	3	4	5
Depth of sample below surface of bar	2.0 ft.	3.0 ft.	2.0 ft.	2.0 ft.
Sample No.	6	7	8	9
Date of collection	8/7	8/7	8/7	8/7
Time	11.45 A. M.	12.30 A. M.	1.55 P. M.	3.30 P. M.
Temperature, in degrees, centigrade	23.0	23.0	25.0	....
Turbidity	0	0	0	5
Free CO_2	30	30	26	9
Dissolved oxygen	1.0	1.0	2.4	....
Chlorine	8	7	10	17
Iron (Fe)	0.5	0.9	1.8	1.0
Alkalinity	44	42	75	136
Incrustants	15	5	0	25
Total hardness	59	47	75	161

The analyses indicated considerable differences in the quality of the water beneath the bar at the different stations. At Station 1, which was near the foot of Neals Island and between the two channels of the river, the ground-water was found to be comparatively hard and of high iron content. This was true also at Station 4, which was 1 000 ft. above the city pumping station. At Station 3, the ground-water obtained from beneath the river was softer than the river water and of low iron content. Near this station, on August 8th, a hole, about 15 ft. in diameter, was dredged in the river bar to a depth of 5 ft. and, in this hole, was placed an 8-in. brass strainer, 8 ft. long, in a horizontal position. The strainer pipe was covered with 18 in. of gravel and about 42 in. of sand. Water was pumped from the strainer by a 2-in. centrifugal pump for the purpose of determining the quality of the water collected in the strainer pipe. Representative analyses of the river water and that obtained from the strainer are shown in Table 2.

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TABLE 2.—PRELIMINARY TESTS OF SMITH SYSTEM AT PARKERSBURG, W. VA., AUGUST 10TH-15TH, 1909.

(Results in parts per million.)

Source of samples	River	Strainer	River	Strainer
Sample No.	10	11	12	13
Date of collection	8/10	8/10	8/15	8/15
Time	10.30 A. M.	10.30 A. M.	9.30 A. M.	9.30 A. M.
Temperature, in degrees, centigrade	26.0	25.0	24.0	23.0
Turbidity	15	0	1 750	0
Free CO_2	.....	.....	2.5	23
Dissolved oxygen	.....	.....	7.0	3.2
Chlorine	56	24	21	17
Iron (Fe)	1.2	0.5	15.0	1.2
Alkalinity	20	24	42	34
Incrustants	110	85	50	15
Total hardness	130	109	92	49
Total solids	.....	.....	749	167
Loss on ignition	.....	.....	58	28
Nitrogen, as nitrites	.....	.....	0.010	0.008
" as nitrates	.....	.....	0	0
Oxygen consumed	.....	.....	9.3	1.8
Bacteria, per cubic centimeter	.....	.....	8 700	82
Coli present in	.....	.....	0.05 c.c.	0.3 c.c.

In general, the water obtained from the strainer pipe was entirely clear, even when the river water had a turbidity of nearly 2 000 parts per million; moreover, the total hardness of the strainer water was less than that of the river water. It was undoubtedly true that the success of this preliminary test interested a large number of citizens and officials at Parkersburg, and showed them the possibility of obtaining a satisfactory water supply in this manner.

At the request of Mr. Gray, still further tests were made on May 7th, 1910, to determine the quality of the water in the bar beneath the river and the feasibility of obtaining a satisfactory supply from horizontal strainers placed beneath the bed of the river. At this time, also, by means of a dredgeboat, the material composing the bar was excavated and examined thoroughly to determine its characteristics at numerous points in the river.

As was true during the first preliminary tests, considerable differences were noted in the character of the water obtained from beneath the river at different depths beneath the surface of the bar and at different locations in the bar. The results of the analyses of the samples collected at this time are shown in Table 3.

After the City had entered into a contract with Mr. Smith to build his system, still further tests were made by the writer in June and July, 1910, for the purpose of determining the best location. More than seventy-five well tests were made at intervals of about 100 ft. over the river bottom, and a small pile-driver was rigged up on a small flatboat to assist in driving the well points. The sampling stations were located by stadia measurements from a base line on

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TABLE 3.—RESULTS OF TESTS OF WATER OBTAINED FROM BENEATH THE OHIO RIVER, BY DRIVING WELL POINTS, AT PARKERSBURG, W. VA., MAY 7TH, 1910.

(Results in parts per million.)

Station	1	1	1	1	2	2
Depth of sample below surface of bar	Surface of river	2.0 ft.	3.5 ft.	5.0 ft.	Surface of river	4 ft.
Sample No.	1	2	3	4	5	6
Time of collection	11 A. M.	11 A. M.	12.20 P. M.	1 P. M.	4 P. M.	4:30 P. M
Temperature, in degrees, centigrade	15.5	14.0	13.3	13.3	.....	13.3
Turbidity	35	50 ±	10 ±	5 ±	40	0 ±
Free CO_2	1	4	4	5	.....	3
Dissolved oxygen	9.1	.....	2.6	0.7	.....	2.2
Chlorine	15	9	9	10	18	18
Iron (total Fe)	4.0	2.0	0.2	0.1	1.5	0.3
Alkalinity	24	128	142	223	20	40
Incrustants	23	20	15	10	27	12
Total hardness	47	148	157	233	47	52
Total solids	176	226	203	289	186	169
Loss on ignition	68	42	43	17	29	46

the bank of the river. By this method of testing, it was learned that the best location for the system proposed was not opposite the Camden Farm, but several thousand feet below this farm at a point where the material composing the river bar was very satisfactory and the underlying water was of low iron content and softer than the river water. Consequently, this location was selected as best adapted for the system.

Further studies of the design of the proposed strainer system indicated the desirability of dividing it into a number of units with independent suction lines, rather than to make only two units, as was contemplated in the original plan. Moreover, on account of the large losses of head which would develop during back-flushing, it was realized that the pumping station should be at the nearest possible point to the strainer system. As shown by the author, the strainer system, as constructed, was composed of five individual units, each connected by separate 18-in. suction lines to the main suction of the pumps, and the latter were near the river bank opposite the center of the strainer system.

It is the writer's belief that two important features of construction have contributed especially to the success of the strainer system, namely, the provision of a deep bed of clean, coarse gravel over the entire area covered by the strainer system, and the reduction in the area of the strainer pipes at their connections to the main distributing pipes to a diameter of $2\frac{1}{2}$ in. The deep gravel layers have increased the capacity of the strainer system materially, and also the reduction in the area of the connections of the strainer pipes has also increased the efficiency of back-flushing the system, by introducing a loss of

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head at this point, thus causing a better distribution of the wash-water.

In view of the possible differences of opinion as to the source, or sources, of the water in the adjacent gravel bed along the Ohio River, and also in the gravel beneath the river, it may be noted, first, that the entire river bottom at Parkersburg is covered to a depth of approximately 18 in. with very hard impervious gravel layers firmly cemented together. In fact, in some places, during the preliminary testing, the gravel layers were so tight that it was found impossible to pump water from the well points after they were driven. This feature is of interest in considering the recommendations contained in one of the preliminary reports to use a hydraulic dredge of such design that it would remove material from the river bar to a depth of 1 or 2 in. Experience in dredging material from the bottom of the Ohio shows that it is impossible to use hydraulic dredges in this river above Portsmouth, Ohio, on account of the compactness of the surface material.

Moreover, some light as to the probable source, or sources, of water obtained at shallow depths from beneath the river may appear from Table 4, showing results of analyses of samples of water collected by the writer at Sistersville, W. Va., on August 1st, 1910.

TABLE 4.—RESULT OF ANALYSES OF SAMPLES OF WATER FROM THE OHIO RIVER AT SISTERSVILLE, W. VA., AUGUST 1ST, 1910.

(Results in parts per million.)

Stations	1	2	3	Ohio River
Depth of water, in feet	4.7	3.9	3.3	....
Temperature, in degrees, centigrade	23.5	23.5	23.0	26.0
Dissolved oxygen	0	0	0	....
Total solids	7 300	6 400	6 100	262
Iron (as Fe)	1 100	1 300	900	0.7
Incrustants	475	482	475	62
Alkalinity*	147	150	156	36
Total hardness	622	632	631	98
Chlorine	4 500	4 250	4 500	27

* Alkalinity, August 3d, 1910, Sample No. 1 = 28; No. 2 = 5; No. 3 = 44.
Samples of ground-water were obtained 4.0 ft. beneath the surface of the bar.

The samples in Table 4 were obtained from the bar at different stations near the center of the river by exactly the same method as that used at Parkersburg, and it is of interest to note that, even at shallow depths beneath the surface of the bar, the ground-water was of remarkably high mineral content. The samples were clear when collected, but in 15 min. they looked like tomato ketchup.

The writer has made a large number of other investigations, along similar lines, to determine the character of the water obtained at shallow depths beneath river bars, principally at Owensboro, Ky., Gallipolis, Ironton, Zanesville, Portsmouth, and Newark, Ohio, and

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Wheeling, W. Va., and, in all cases, the results obtained have been generally similar to those at Parkersburg. The tests have indicated great variations in the quality of the water at different locations and at different depths of sampling. The writer's conclusion is that the underground water, even at shallow depths, is always of very different quality from the river water, especially during low stages of the streams; also, that during low stages it cannot have its source in the river, but must come from adjacent higher territory and flow toward the river.

An interesting feature was developed at Parkersburg where, in some cases, the water obtained from the well points was of very high iron content and contained only 5 parts of incrustants when analyzed by the usual methods. Its alkalinity was 55 parts. Two or three such points were included within the area covered by the strainer system, and excavations disclosed the fact that a log or trunk of an old tree had been deposited and covered within the river bar. Immediately around these logs, for distances of about 10 ft., the surrounding gravel and sand was filled with iron. When the log and iron deposits were removed, the quality of the water obtained from the ground changed, and resembled that flowing from adjacent areas. Its iron content was reduced to about 0.3 part per million, the incrustants increased to about 70 parts, and the alkalinity was reduced to about 40 parts. The writer has no explanation for this phenomenon.

In that part of the report of Messrs. Fuertes and Fuller which discusses the quality of the ground-water obtained from the test wells it is stated that, in some instances, the ground-water contained objectionable quantities of iron "due to the water dissolving iron during the long intervals of standing in contact with the metal;" and, again, that the large quantity of iron in the well water at the steel plant is explained "partly by the flow of water through a long line of pipe." The writer believes that these statements should not go unquestioned, because undoubtedly the high iron content of the ground-water was not attributable to its contact with iron pipes, but the iron was present in the water in the gravel strata. It is significant that the test wells producing water of high iron content were opposite those points in the river where the well point tests indicated that the underground water beneath the river, also, was of high iron content. It is a common popular fallacy to attribute the iron found in some well waters to their contact with the pipes.

Moreover, the writer would question the judgment which dictated placing the wells along the river bank 400 ft. apart. Such a spacing would seem to be too great, and would not make available all the water contained in the gravel strata. Although the water levels in the test wells indicated that the contours were parallel to the river bank, it

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should be noted that these levels were taken during comparatively high stages of the river. It is the writer's opinion that if levels had been taken during low stages, it would have been found that the ground-water table had a different slope, and that the flow of water would have been slightly down stream toward the river bank. This would conform more nearly to the conclusions of Mr. Clapp, as stated in his report.

It is believed that experience at Parkersburg warrants the conclusion that each of the following features is of importance and should be considered in the construction of strainer systems or infiltration plants of this type:

1.—The presence of a permanent bar of sand and gravel of suitable size within a reasonable distance of the pumping station to which the water is to be carried;
2.—The constant submergence of the bar from which the supply is to be obtained;
3.—The velocity of flow in the stream, which velocity must be sufficient to maintain the bar clean and free from silt and other drift;
4.—The depth at which the collecting units are placed with relation to the bottom of the stream; and
5.—The character of the water which is found beneath the bottom of the stream.

Experience at Parkersburg has shown, also, that it is very important that there should be a suitable meter or measuring device on each suction line from each section of the system. Without such meters, it is impossible to operate the different sections of the system at the same rate. Immediately after back-flushing a section, the loss of head is very much decreased, and it is difficult to avoid excessive rates of filtration from the newly washed unit without a suitable meter.

Although this paper was presented primarily with a view of discussing the preliminary investigations by engineers to determine the merits of the various water supplies available at Parkersburg, it may be well to mention briefly the merits of the system actually built. Its obvious advantages are its low cost of operation and its simplicity and ease of operation. The annual cost of operation is only one-half of the estimated cost of a suitable well system or rapid sand filter plant, as discussed in the preliminary reports of the consulting engineers. Moreover, the operation of the plant is accomplished successfully without expert control.

Though it may be true that the results accomplished by the plant are not within certain arbitrary standards adopted by Federal authorities, it is to be noted that the plant was constructed under more severe

guaranties of efficiency than have ever been enforced at any other water purification plant in the United States. Moreover, the plant was accepted on the basis of these guaranties and on the basis of thorough tests of operation by a competent representative of the City. Mr. Burgess.

It is the writer's belief that the City should erect a suitable chlorinating plant as an additional precaution in securing always an entirely satisfactory supply. In doing so, however, it should be recognized that the City is taking no additional precautions other than those now adopted by very nearly all cities which obtain their water supplies from either rapid or slow sand filters. There is no doubt that the results obtained by the present plant are comparable with those from rapid sand filters, and that the cost of operation is so low that it makes the type now in use at Parkersburg of very great advantage to any city where conditions are such as to permit of its satisfactory installation.

JAMES H. FUERTES,* M. AM. SOC. C. E.—Messrs. Fuller, Hill and Potter, having discussed quite fully the principal and more important matters dealt with by Mr. Hall, a few remarks concerning experiences with somewhat similar methods of securing water from river beds may be of interest. Mr. Fuertes.

A number of water companies and cities on the Allegheny and Ohio Rivers in the vicinity of Pittsburgh take water from these rivers by a plan having general features in common with those of the plan adopted at Parkersburg.

About 10 years ago three citizens of Wilkinsburg, Pa., a suburb of Pittsburgh, brought suit against the Pennsylvania Water Company to compel that company to furnish pure water, as required by the terms of its charter. The suit was rather unique, and was one of the few recorded in which a decree has been entered directing a company to secure and provide a sufficient supply of pure and wholesome water. The supply against which complaint was made was drawn from gravel-covered cribs in the Allegheny River bottom. The first supply crib, built in 1897, was 32 ft. wide, 308 ft. long, and 5 ft. high, made of 2 by 6-in. and 2 by 8-in. planks, placed edgewise and flatwise, and separated by intervals of 2 in. A 24-in. pipe extended from the interior of the crib to the pump-well on the shore. The crib was sunk in a dredged excavation, about 300 ft. from the north shore of the river, and at a depth placing the top of the crib about 5 ft. below the natural bed of the river. The excavation was refilled and the crib covered to a depth of 1 ft., with 2-in. gravel, and then with from 4 to 4½ ft. of sand and fine gravel, such as had passed through a 1½-in. mesh screen. The depth of the water over the crib was ordinarily from 8 to 10 ft.

* New York City.

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It was shown by analysis of the water that the supply was drawn partly from the river by filtration through the gravel over the cribs, and partly from the sub-stratum of ground-water in the 30-ft. bed of gravel and sand in the river bottom.

In 1902, two additional cribs, each 48 ft. wide, 408 ft. long, and 5 ft. deep, were put in service. They were constructed in the same general manner as the first, but excluded from the gravel covering stones larger than 1 in. in diameter. Three cribs were sunk side by side and 10 ft. apart, and a line of 42-in. pipe, decreasing to 24-in. in diameter, was laid in the 10-ft. space between the cribs, with four short pipe connections therefrom to each crib about equally spaced throughout their length. The main 42-in. pipe was extended under the river bottom to the pump-well on shore.

The water from the cribs discharged under its own head into the pump-well, and was pumped thence to a reservoir, from which the street mains distributed it to the consumers.

After periods of operation of varying lengths, the surfaces of the gravel over the sunken cribs would become more or less choked with silt, dirt, and other matters suspended in the river water, and gradually fail to yield a sufficient supply. To relieve this shortage, the company had recourse to cleaning or washing the filter areas with water. The filtering area immediately over the cribs—with all three cribs in use—was about 1.13 acres; undoubtedly, however, water entered the cribs from the bottom and from areas outside of the boundary lines of the cribs. It is not possible, therefore, to state the rate of filtration per unit of area per day, except that it was less than about 7 000 000 gal. per acre per day by the quantity derived from sources other than the filtering area over the cribs.

The plan followed in cleaning these filter areas was to drag back and forth across the area a rake having a 2-in. pipe header, 3 ft. long with seven teeth, 15 in. long, each perforated with two $\frac{3}{16}$-in. holes on the front and one in the end, water being forced down through this rake at a pressure of 125 lb. per sq. in. by a pump on a barge. The rake was dragged across this area, the wash-water rising up through the sand and bringing the mud with it, the principle resembling that by which the Blaisdell filter washer is operated. The river current was depended on to carry the washings down stream and away from the washed areas. The rake was dragged back and forth across each area three times, by hand windlasses on barges anchored on each side of the areas being washed, before a new area was attacked.

Raking, or scouring, was practically continuous from May to November, 1905, and from May to December, 1906, except as interfered with by uncontrollable conditions, such as river floods. In 1906, the surface was examined several times by a diver, who reported

considerable disturbances of the sand surface, and, under his guidance, depressions were filled up with gravel and sand deposited from a barge through a tube. Frequently, the diver rode on the rake, as it was dragged back and forth, to press the teeth down into the sand. It was stated that the teeth usually penetrated the top of the gravel bed to a depth of from 8 to 12 in. While the process of raking or scouring was going on, the cribs were kept in constant use. The disturbance of the surface and the destruction of the silt formation thereon permitted a largely increased volume of water to pass freely through to the cribs, through the washed areas, and during and immediately after these periods of washing, the water was easily shown to be very little different, in character, from the raw river water. Mr. Fuertes.

Much evidence was introduced at the trial as to the quality of the water, both as to its chemical and biological features, and it was shown that there were times when, for a number of consecutive days, the quality of the water would be excellent; all of a sudden, however, some change in conditions of river flow would cause the silt to be washed from the river-bed over the cribs, or parts of their areas, leaving places where the river water could penetrate to the interior of the cribs in a practically unchanged and polluted condition.

In 1905 and 1906, typhoid fever in Wilkinsburg became epidemic, and just prior to the bringing of the suit it had the highest typhoid fever case and death rate of any town in the United States. Very heavy rates also prevailed in the 37th Ward of Pittsburgh, and in Swissvale Borough, also supplied with water by the same company. As a result of this alarming condition, protests were lodged, and, finally, recourse was had to the Courts to compel the Water Company to safeguard adequately the quality of the water. The case, recorded as E. Z. Peffer *et al. vs.* Pennsylvania Water Company, was No. 396, tried in the August term, 1906, of the Court of Common Pleas, No. 3, of Allegheny County, Pennsylvania, Miller, J., specially presiding. The conclusions of law, in this case, as drawn by Judge Miller, were:

First.—The fact having been found that the water complained of, as furnished by the defendant, is not reasonably pure and wholesome, it follows that the plaintiff's bill must be sustained.

Second.—An interlocutory decree will be entered, directing it to secure and provide a sufficient supply of pure and wholesome water; further directing that it shall, within three months from the date of decree, file a statement of the steps it has taken, and purposes to take, in compliance with the requirements to furnish reasonably pure and wholesome water, upon the submission of which the plaintiff may file a reply or answer, as he may deem advisable. It will be further decreed that this case be retained for such further proceedings

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as may be necessary to insure its performance, and to enable the Court to exercise the jurisdiction conferred by this Act of Assembly under which this action is brought, with liberty on the part of either of the parties to apply to Court for further orders and decrees as may be necessary and just.

Third.—The defendant to pay the costs.

The meagerness of the data regarding the details of design, construction, and operation of the Parkersburg plant, in Mr. Hall's paper, makes extended discussion of it impossible. Those elements of design which would give control over the process of filtration are scarcely mentioned. Enough is stated, however, as to the diameters of the main pipes leading from the strainer system, and as to the sizes of the laterals and their perforations to indicate:

1.—That hydraulic conditions must exist which would be avoided in a properly designed filter plant.

2.—That there can be almost no control over the effectiveness of back-flushing, or washing of the filter areas.

3.—That the loss of head must vary between quite wide limits at various parts of these areas during filtration, and after partial clogging shall have taken place, and that, therefore, parts of the areas must be passing the water at much higher rates than others, and, consequently, must be doing less effective work than planned.

4.—It is also evident that the back-flushing is likely to blow up and overwash parts of the areas, and leave other parts badly clogged.

5.—That floods in the river may scour, and probably have frequently scoured, holes in the filters—possibly almost down to the strainers.

6.—That the whole filter area is inaccessible, and utterly beyond inspection and repair during floods, the times when the water is worst and the need of protection greatest.

The Parkersburg plant may be expected to give fair water when the river is in its best condition, and, at such times, it is almost good enough to use without filtration. When the river water is turbid and dangerously polluted, the water delivered by the plant may or may not be safe. Whether or not it is safe will be beyond the knowledge of the men operating the works; and, if it is unsafe, it is entirely beyond their power to make it so by any agency or device inherent to or part of the plant itself or of its design.

Judging by the discoveries of Mr. Leland, when making the inspections in September, 1916, referred to by Mr. Hall, the Parkersburg plant is experiencing difficulties similar to those which led

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ultimately to the virtual condemnation of the Wilkinsburg works and to the erection of a properly built mechanical filter plant in its stead.

The cost of construction and operation of the works, as built at Parkersburg, are given by Mr. Hall as $284 171.22 and $11 892.46, respectively. The annual cost was based on the pumpage of an average of 3 000 000 gal. daily. In the Fuertes-Fuller report, the costs were given as $165 945 and $24 000, respectively, the latter figure being the estimated annual cost of operation for a daily yield of 4 000 000 gal. Reduced to the basis of 3 000 000 gal. daily, for comparison with the figures given by Mr. Hall for the Parkersburg plant, and adding to the cost of construction certain items included by Mr. Hall, and with interest at 7% in each case, the annual operating and maintenance costs of the two plants would be practically equal. The Smith system, therefore, has required at Parkersburg a greater investment, saves nothing in annual expense, and gives to the city far less security against water-carried diseases than would have resulted from the establishment of a properly built and operated mechanical filter plant, such as recommended and described in the Fuertes-Fuller report.

Mr. Hall gives, among the reasons advanced by Mr. Gray for the adoption of the Smith system for Parkersburg, that,

"If properly built and operated, this system will furnish water of a better quality than a mechanical filter plant, as regards steam-raising purposes, on account of the slight increase in permanent hardness which is caused by the use of a coagulant."

Although it is true that the use of aluminum sulphate as a coagulant will result in the conversion of a portion of the alkalinity into the sulphate or incrustant form of hardness, an examination of the analyses in Appendix I shows that throughout January, 1914, the total hardness of the water from the Smith system was about 87 parts per million, as against about 74 parts per million for the river water, this increase being due to the increased alkalinity of the sub-river ground-water. The increased quantity of soft scale due to the extra 13 parts per million of hardness of this ground-water is probably fully as disadvantageous from a steam-raising point of view as would be the conversion of about 5 or 6 parts per million of the low alkalinity of the river water into a proportional quantity of incrustants by the application of aluminum sulphate used in connection with mechanical filters. For summer conditions, when both the river water and the water secured by the Smith system would be much harder than reported in the January analyses, the advantages would probably lie strongly with the mechanically filtered water.

And, further, with the mechanical filters, the operatives would have access to and control over the filters and their efficiency at all

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times. With the Smith system, the whole delicate process of filtration is beyond reach, often buried under floods and subject to disturbance by wash and scour, or to irregular deposition of mud on the filter area, with consequent imperfect purification, due to varying and excessive rates of filtration on parts of the area smaller than the total required.

Taking everything into account, the speaker is not convinced, by the data brought forward, that the proper decision was made at Parkersburg when the Smith system was adopted in preference to a properly designed and constructed mechanical filter plant.

It is much to be regretted that so little information has been furnished as to the operation of and results obtained by the Parkersburg water-works plant. Fancied security has led to neglect of vigilance, as represented by systematic, continuous, analytical studies of the river and secured waters, and more frequent examinations of this nature, as recommended by Mr. Hall, should certainly be made by the authorities.

The ground-water studies, made at Parkersburg, and referred to by Mr. Hall, show a number of interesting facts. Figs. 3 to 12 were selected from among 32 such diagrams in the Fuertes-Fuller report, and seem to have been well chosen to exhibit the interesting phenomena of filling and emptying the subterranean storage spaces in the voids in the gravel underlying the flat lands along the river north of Parkersburg.

It will be noticed that the water finds its way into the ground, from the river, with comparative difficulty, as judged by the steep slope of the ground-water in the neighborhood of the river banks, but that it makes its escape back into the river, when the latter falls, with comparative freedom. This is as would be expected, the surface of the river bottom and banks having become clogged with mud (strained out of the river water as it filtered out through the banks and bottom) on a rising river, but being cleansed, or washed free of mud to a greater or lesser extent, on a falling river, by the passage of the ground-water back again into the river.

The contours on Figs. 16, 17, and 18 show the gradual change in direction of the flow of the ground-water away from and toward the river above Parkersburg. Fig. 12, which was reproduced from among the diagrams in the Fuertes-Fuller report, occupies a position between Figs. 17 and 18. An examination of these diagrams shows the following interesting conditions.

On January 13th (Fig. 16) the river, after having reached its lowest point from a previous rise, was just beginning another rise which, by the 15th, had reached 9 ft. Observing the ground-water contours on this diagram, it will be seen that the general trend of the ground-water was down stream and toward the Ohio River,

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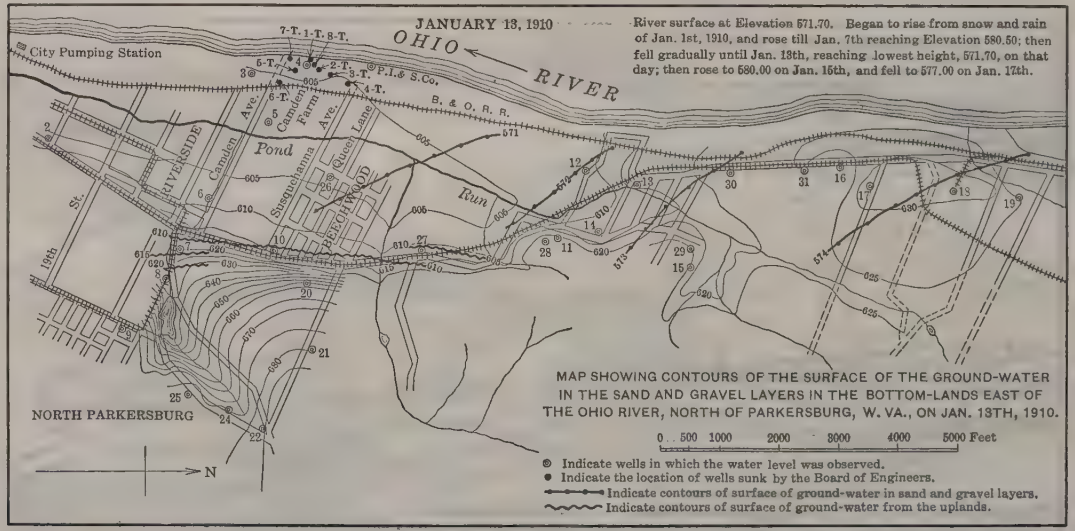
JANUARY 13, 1910
River surface at Elevation 571.70. Began to rise from snow and rain of Jan. 1st, 1910, and rose till Jan. 7th reaching Elevation 580.50; then fell gradually until Jan. 13th, reaching lowest height, 571.70, on that day; then rose to 580.00 on Jan. 15th, and fell to 577.00 on Jan. 17th.
OHIO RIVER
City Pumping Station
P. I. & S. Co.
B. & O. R. R.
Pond
Run
RIVERSIDE
Camden Ave.
Camden Farm Ave.
Queen Lane
Susquehanna St.
BEECHWOOD
NORTH PARKERSBURG
MAP SHOWING CONTOURS OF THE SURFACE OF THE GROUND-WATER IN THE SAND AND GRAVEL LAYERS IN THE BOTTOM-LANDS EAST OF THE OHIO RIVER, NORTH OF PARKERSBURG, W. VA., ON JAN. 13TH, 1910.
0 500 1000 2000 3000 4000 5000 Feet
Indicate wells in which the water level was observed.
Indicate the location of wells sunk by the Board of Engineers.
Indicate contours of surface of ground-water in sand and gravel layers.
Indicate contours of surface of ground-water from the uplands.
N

FIG. 16.

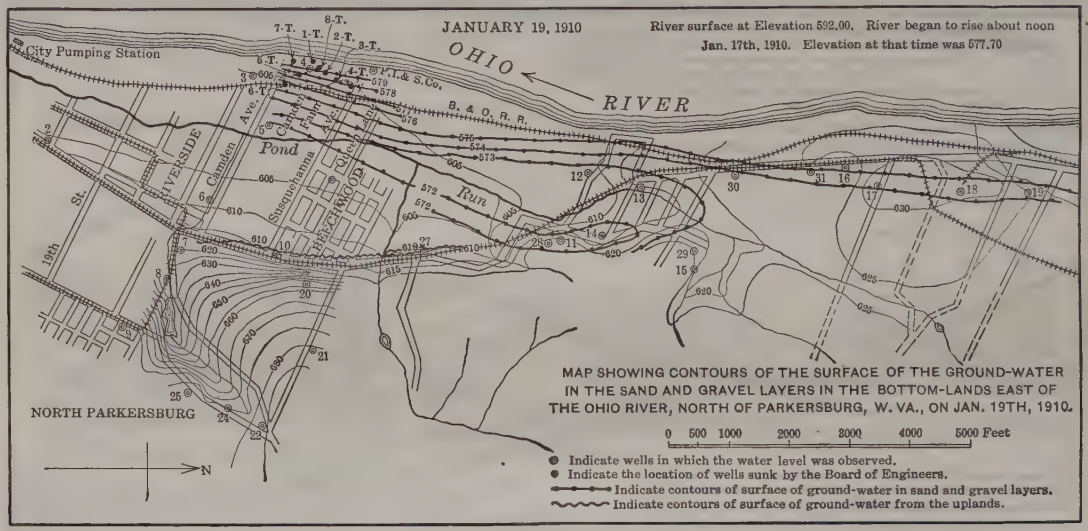
JANUARY 19, 1910
River surface at Elevation 592.00. River began to rise about noon Jan. 17th, 1910. Elevation at that time was 577.70
City Pumping Station
OHIO RIVER
B. & O. R. R.
Pond
Run
RIVERSIDE
Camden Ave.
Susquehanna Ave.
Chesapeake Ave.
Shenandoah Ave.
19th St.
NORTH PARKERSBURG
MAP SHOWING CONTOURS OF THE SURFACE OF THE GROUND-WATER IN THE SAND AND GRAVEL LAYERS IN THE BOTTOM-LANDS EAST OF THE OHIO RIVER, NORTH OF PARKERSBURG, W. VA., ON JAN. 19TH, 1910.
0 500 1000 2000 3000 4000 5000 Feet
Indicate wells in which the water level was observed.
Indicate the location of wells sunk by the Board of Engineers.
Indicate contours of surface of ground-water in sand and gravel layers.
Indicate contours of surface of ground-water from the uplands.
N

FIG. 17.

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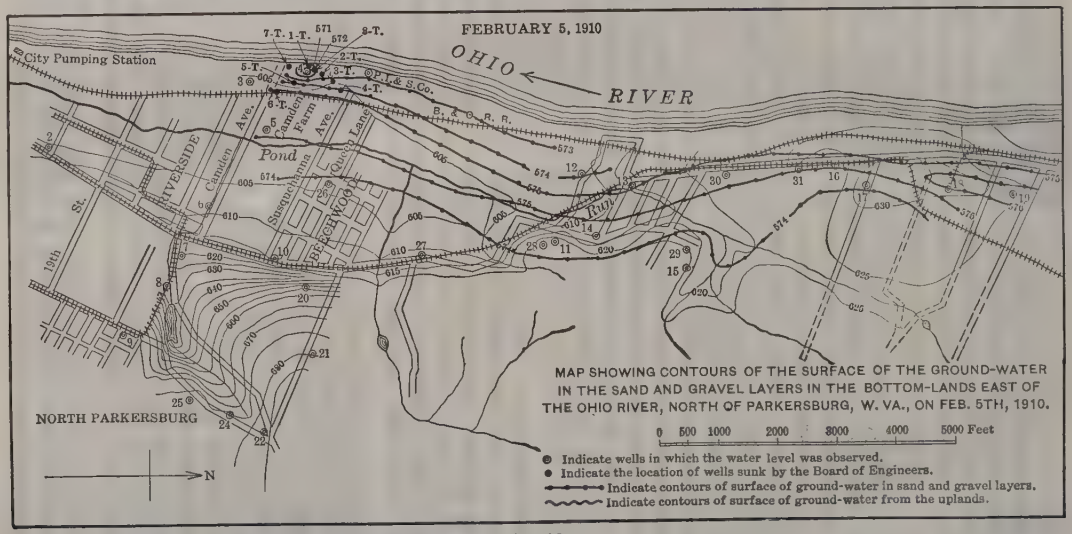
FEBRUARY 5, 1910
OHIO RIVER
City Pumping Station
P. L. & S. Co.
B. & O. R. R.
Camden Ave.
Farm Ave.
Queen Lane
Pond
Susquehanna
RIVERSIDE
BELLEVUE
St.
19th
Run
NORTH PARKERSBURG
N
MAP SHOWING CONTOURS OF THE SURFACE OF THE GROUND-WATER IN THE SAND AND GRAVEL LAYERS IN THE BOTTOM-LANDS EAST OF THE OHIO RIVER, NORTH OF PARKERSBURG, W. VA., ON FEB. 5TH, 1910.
0 500 1000 2000 3000 4000 5000 Feet
Indicate wells in which the water level was observed.
Indicate the location of wells sunk by the Board of Engineers.
Indicate contours of surface of ground-water in sand and gravel layers.
Indicate contours of surface of ground-water from the uplands.

FIG. 18.

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the 571-ft. contour being the farthest down stream and the 574-ft. one the farthest up stream, all these contours lying in approximately parallel lines, and indicating a motion diagonally down stream toward the river.

On January 17th, the river began another rise, and, within the next few days, reached an elevation of 597.5 ft. On January 19th (Fig. 17) it stood at Elevation 592. It will be observed that the contours of the surface of the ground-water had changed entirely, swinging around parallel to the shore with the highest contours (Elevation 579) along the river bank, and the lowest still at Elevation 572, on a line parallel with and about 2 000 ft. distant from the river bank. By the 24th, the river surface had fallen to 692.5 and the ground-water near the river bank had risen to 584, or 14 ft. higher than on January 13th, whereas, along a line practically parallel with, and some 2 500 ft. east of, the river bank, the ground-water level had risen only about 1 ft.

On February 5th (Fig. 18) the river still continued to fall, the ground-water which had been piled up to Elevation 584 on January 24th had fallen at the river bank to an elevation of only 573, gradually rising to an elevation of 575, about 2 000 ft. back from the river and dropping to 574 a few hundred feet farther east. In this case the water was slowly flowing in both directions from a line parallel to and about 2 000 ft. back from the river bank.

It is very fortunate that this flood occurred during the time that the tests were being carried on at Parkersburg, as the occasions on which such phenomena can be observed are relatively rare.

Mr. Fuller has made the suggestion that in a short time the results of studies for the more complete identification of certain forms of bacteria will be made public. The present laboratory methods for the quick determination of the probable presence of *B. coli* in water, are, as Mr. Fuller has said, not only far from satisfactory, but actually misleading under some conditions. An accurate and extensive knowledge of the history of the water is always essential in determining the likelihood of the presence of these organisms, if shown to be probably present by laboratory tests. The speaker calls to mind an interesting experience of this sort in connection with the examination of the water supply of the Insane Asylum at Willard, N. Y., in December, 1901, and January, 1902.

Diphtheria had been epidemic at the Willard Asylum since 1897, and all efforts to eradicate it from the institution had proven unavailing. The cases were confined mostly to the nurses, physicians, and attendants, very few cases being noted among the inmates. The superintendent of the asylum and his wife were both in quarantine with diphtheria while this investigation was going on. In connection with the various studies made to determine the cause of the prevalence of

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the disease, a thorough examination was made of the water supply and sewerage of the institution. In order to eliminate possible sources of error, a great many samples of water, from widely separated locations, were collected and examined.

The water supply for the institution was taken from Seneca Lake at a point some 400 or 500 ft. from shore, where the water was about 30 ft. deep, and at a point about 1 800 ft. from where the main sewer of the institution discharged into the lake.

An apparatus was devised for securing samples of lake water at various depths, and these, after collection, were put in sterilized, glass stoppered bottles, and examined promptly.

Samples were taken at the surface of the lake and at various depths at numerous points, some of these being in the immediate vicinity of the sewer, some at the water-works intake, others on the opposite side of the lake, about 1½ miles from the intake, others at the ends of the lake, more than 20 miles distant. The samples were collected at various depths, from the surface to 300 ft. below the surface.

These analyses disclosed the interesting fact that the chlorine content of the lake water varied from 36 to 37 parts per million, which was about one-third of the quantity that might be expected in ordinary town sewage. Organisms giving the reactions expected of *B. coli* were found in the samples collected in the neighborhood of the sewer, also, in samples collected 50 ft. below the surface on the other side of the lake and about 600 ft. from shore, also, in some collected 50 ft. below the surface, some 600 ft. off shore about 2 miles away, and in a number of other places. Previous analyses made in 1899 had disclosed the suspected presence of *B. coli communis* in samples 1 300 ft. out from shore, on the line of the hospital's sewer, at a depth of 20 ft.; ½ mile from shore at a depth of 150 ft.; ½ mile from shore on a line with the water-works intake at a depth of 400 ft.; 500 ft. deep off shore about 5 miles south of the Asylum, and in the center of the lake at a depth of 400 ft.

In the analyses made in 1901-02, it was pretty well established that the organisms giving the reactions for the differentiation of the *B. coli communis* were probably washed into the lake from the cultivated fields along the sides, and bore a close relation to the rust on grain.

The high chlorine in the lake water was caused by the presence of underground salt beds in that vicinity; the normal chlorine for that region would not be more than 7 parts per million.

Without knowing all the conditions, the analyses of Seneca Lake water might easily have been mistaken for a highly sewage-polluted

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water; a conclusion which the most ordinary common sense would refuse to accept.

More positive methods for identification of sewage bacteria in waters have long been needed, and water-works engineers will welcome the greater certainty promised by Mr. Fuller's predictions.

Messrs. Knowles and Stevenson.

MORRIS KNOWLES,* M. AM. SOC. C. E., and J. D. STEVENSON,* ASSOC. M. AM. SOC. C. E. (by letter).—This paper is of especial interest to the writers, because of the part they took, in 1908-09, as Consulting and Resident Engineers, respectively, of the Water-Works Commission, in the early studies of the water supply at Parkersburg and the proposed new systems. They believe that a statement of some of the considerations which led to the recommendation to develop a well supply, instead of the system described by the author, will be of interest. The author is to be commended for collating and bringing before the Profession the various studies and reports which have been made in reference to this interesting situation.

No discussion of this subject would be complete without mention of the lively interest taken in these early investigations by the late United States Senator J. N. Camden, and continued, at his death, by his estate. The generous attitude of this public-spirited citizen can be best stated by quoting from an open letter from him to the city officials, as follows:

"I, therefore, respectfully propose, in order to test the question of natural filtration, that I will, at my own cost and expense, sink and test wells, both on the river front and on the foot of Neal's Island, for which I already have the permission of the owner, to ascertain the quality and quantity of water that can be obtained by natural filtration, and to commence as soon as the engineers selected by the city authorities and water-works commission are ready to direct and superintend the tests to be made. This will cost the city nothing and will no doubt be of value in arriving at conclusions. I will also add that, should the city desire to locate its plant or pumping station or wells upon any ground owned by me, I will donate to the City all the ground it may need for water-works purposes."

In pursuance of the engagement of the writers as engineers, a report was made on August 24th, 1908, covering an investigation of ten possible developments and including a recommendation that either a mechanical filter plant or a system of wells be adopted as a source of supply. A definite choice between these two projects was not recommended at that time for two reasons: First, because the original sum of money appropriated to the Commission did not permit of drilling and testing wells; and, second, because after the money donated by Senator Camden for this purpose was made available, it was concluded "that the time was not propitious for either drilling

* Pittsburgh, Pa.

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or testing wells, as both the river and the ground-water level were too high to justify any conclusion from a test as to safe yield."

The investigation of the wells was only postponed, however, being carried out in the succeeding warm, dry season, at a time when both surface and ground-water levels were low. The conditions at the time of the tests were most favorable to a safe conclusion, as the river was lower than it had been for many seasons. The results of these tests warranted the recommendation of a water supply to be drawn from wells on the Camden Farm, which was reported on January 25th, 1909.

The topography, geology, and history of the Ohio Valley give evidence that, at one time, either the Little Kanawha River or Worthington Creek, or both, flowed through a gap in the hills and into the Ohio River at a point near the present settlement of Beechwood. In later years, either the Little Kanawha River straightened its course and shifted to its present location, while Worthington Creek occupied in reverse flow what had been the Little Kanawha River bed; or else, Worthington Creek cut a new course into the Little Kanawha River and thence into the Ohio. In any case, the gap in the hills became filled with sand and gravel, forming what are now known locally as the "Sand Plains", separated from Parkersburg by the rocky promontory known as Boremans Hill.

Evidence is conclusive that in times past the Ohio skirted the West Virginia shore and, at one time, occupied the area now forming the east shore terraces, on which are the Camden Farm and the settlements of Beechwood, Vienna, and Brisco. It is also evident by the slope of the rock that, in its former location, the Ohio River was at a lower elevation than at present, the intervening space between the present and the former beds having been filled with a water-bearing sand and gravel formation. The trough made by the intersection of the rock beds sloping from the two shores is under the east shore terraces, and, at the Camden Farm and for many miles up stream, probably has the same general course as Pond Run.

The flow of underground water normally follows the general inclination of the rock, and is outward from the surrounding hills. When the flows from two hills meet, the resultant flow is upward, and if it is abundant and the head is sufficient, an Artesian flow will result. Such is the origin of much of the flow in Pond Run. An inspection of the bottom and sides of the Run and feeders will disclose many springs, and these have been found from above Neal's Island to a point below the Camden Farm.

It would not be expected that there would be a large flow parallel to the river in the sand and gravels directly over the rock, but rather that something like an underground reservoir would be formed, over-

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flowing into the river and to the surface of the terrace. However, in this particular case, the gravel beds are topped by impervious clay and loam. As a result, the head due to the higher level of the ground-water on the "Sand Plains" causes flow in the direction of the inclining floor of the gravel beds, which is not only toward the river, but also down the valley. This was the basis for the proposed development by two lines of wells, one parallel to, and another at right angles to, the river, thus intercepting whatever water flowed toward or parallel with the river.

It will be noted that the top of the gravel beds at the river's edge is very little higher than the low-water level in the river. The observations made by the writers indicate that the underground reservoir is not fed to any great extent by infiltration from the river, but probably receives most of its water from rainfall on the terraces, on the "Sand Plains", and from the 26 sq. miles of drainage area tributary to Worthington Creek, and probably some from entrance at the rock riffles in the Ohio some distance up stream.

In Figs. 3 to 10, inclusive, it will be noted that the surface of the ground-water as shown is always above the line of demarcation between the water-bearing sand and gravel and the impervious loam and clay; and that in no instance did the pumping lower the surface of the ground-water into the sand and gravel. It may be concluded, therefore, that, even at high stages of the river, no large quantity of river water would be forced into the sand and gravel; for these strata would always be filled, and, as the impervious clay and loam above would not absorb water in quantities rapidly, the rise in the river would only increase the head on the water in the sand and gravel, raising the level of the water in the observation wells correspondingly.

Referring to Fig. 11, the following explanation appears in the writers' report:

"When pumping was checked in amount or stopped entirely, the water quickly rose in the well being pumped, indicating a lack of strainer openings for the free access of the water."

In the absence of long-continued tests, the quantity of water which may be procured from any formation is largely a matter of speculation. The planning of a water supply from wells in any undeveloped territory, therefore, should provide, where possible, for permitting easy future adaptation to other sources of supply, in case this should become necessary. The conditions on the Camden Farm were particularly advantageous for such a water-works development; and, although, as a result of the tests and the comparison of conditions with those in somewhat similar deposits elsewhere, the conclusion reached by the writers, that wells, properly located, drilled, and developed, would yield continuously, for a sufficient period, all the water

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to be reasonably demanded by the City of Parkersburg; at the same time, the various units could be designed so as to be readily adapted, with a minimum monetary loss, to use in connection with a filter plant, to provide against the remote possibility of failure of the wells.

The estimated cost of the construction recommended by the writers was $220 865. Of this sum, 54% was for distributing reservoir, reinforcing mains, etc., and would be equally useful in connection with the water-works then existing, or with any other that might be designed; 33% was for pumping stations and equipment—structures equally useful, either with a filtration plant, or with a well supply; and only 13% was allotted to equipment useful for a well supply alone. Thus, it is evident that, even if the wells were to be abandoned at some future time, the loss on the investment would be trivial, and fully met in such times by the lesser cost of operating the well system.

Among the various developments studied in connection with the writers' investigation at Parkersburg was that known as the Smith infiltration system. The four principal reasons advanced at that time by the promoters for its adoption were: Economy in installation and operation; purified river water and not hard ground-water; ease of repairs and replacements; and reliability of supply. Instead of confirming any one of these points, the investigation of the writers decidedly discredited all of them and, further, led to the conclusions: that the infiltration system, being submerged in the bed of the river, is subjected to the shifting of the river bottom, and to uncontrolled variation in the thickness of the covering over the strainers; that such variation, whether increase or decrease, would be detrimental to the process, and that the infiltration system, being below the low-water level, cannot be subjected to the same accurate control and careful observations and studies of operation as are possible with a modern filter; furthermore, that the only method of cleaning the beds is by back-flushing, and that such a procedure, without controlling devices for guaranteeing an even distribution of the wash-water, may do decidedly more harm than good; and, finally, that the danger of the introduction of polluted river water into the system was ever present. A special report, dated January 15th, 1909, therefore, recommended that it was unwise to introduce such a system. It is now evident that several of the difficulties foreseen have already occurred; the author's account of the replacements during the summer of 1916 point to this fact.

The investigations of the Pittsburgh Filtration Commission in 1907-09, with which one of the writers was connected, showed that no system of filtration, without a sufficient period of subsidence, will effectively treat the Allegheny River water; and that, if the rate of filtration is increased to more than the normal moderate rate of slow sand filtration, a coagulant must be used. The investigations at Cin-

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cinnati fixed an average of 125 parts per million as a conservative estimate of the suspended matter in the unsubsided Ohio River water, which could be treated regularly and fairly satisfactorily by slow sand filters. On this basis, a slow sand filter without a period of subsidence would be a failure 65% of the time, if treating this river water.

For all these reasons and because of the many points of inferiority of the Smith infiltration system, when compared with the modern slow sand filter, designed, constructed, and operated in accordance with accepted engineering practice, a negative recommendation on the Smith system was made.

There is, however, nothing new or novel in the Smith infiltration system. Although it was claimed to be an improvement on and substitute for the established method of slow sand filtration, in reality it resembles more nearly, both in principle and in practice, the "filter cribs" which have been used for the supply of water to communities along the Allegheny and Upper Ohio Rivers, and, no doubt, on many other streams, for more than 20 years. In most cases, the construction of the latter consists of a large wooden crib, open at the sides and bottom, placed in an excavation in the river bed, and covered on top with from 5 to 7 ft. of selected sand and gravel.

At the time of the investigations, by the Pittsburgh Filtration Commission, of possible sources of water supply for that city, considerable study was given to the results obtained through filter cribs. Analyses of the river water and of effluents from the cribs indicated that, in general, the effluents from the cribs were clear and free from suspended matter, except at extreme high stages of the river, and that the number of bacteria in the effluents were usually less, but the hardness materially greater, than that of the water from the river itself. The Pittsburgh investigations showed clearly that, early in the life of the cribs, much or all of the water is drawn from the sand and gravel beds lying under and around them; but that, with age, the sand and gravel beds become clogged, and larger and larger quantities of water are drawn down from the river through the top covering. In many cases this downward flow is increased by raking the sand covering, and although the top covering may be broken up in this way or even replaced, it is impossible to clean efficiently the material under and around the crib. With the increased infiltration direct from the river through the top, there is a corresponding deterioration in quality. These statements are fully confirmed by the experience of the numerous water plants with crib supplies which have been obliged to supplement these with filter plants within the past few years.

The author has expressed the opinion that a large percentage of the water drawn is taken from the sand beds, possibly a larger part than that from the open river. This opinion is quite in accord with the experience of the writers, and it would appear not unlikely that

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during the early life, while the strainers are new, and before the strata at the bottom and sides become clogged, 70% or more of the total water may be drawn from this lower bed. The so-called filtered water, therefore, is in large part a ground-water, similar in chemical characteristics to that which would be had from wells; and in so far as satisfactory results with respect to quality are secured, they are without doubt due more to the quality of the ground-water than to the efficient filtration of the river water. Nor would it be safe either to predict how long the situation might continue, or to expect other than unsatisfactory results with the increase of the proportion of river water. Back-flushing, if forceful enough to be successful, invariably results in disturbing the sand layers to a dangerous degree and frequently forms craters or troughs of washed material, leaving the rest of the bed undisturbed and uncleansed.

It is worthy of note also that, at Parkersburg, the decision between a well supply and a supply by mechanical filtration was a very close one. Either system was considered certain to provide a safe, abundant water supply for the community, and both systems had the endorsement of the best engineering practice and were sanctioned by the most exacting Boards of Health in adjoining States. The system adopted had none of these assurances, and was closely akin to a practice which has been condemned, and is rapidly being abandoned in neighboring States, for water supply for domestic purposes. It would seem that the time is fast approaching when most of the expenditure of $80 700 will be required to be renewed or replaced by some other system.

If the story of the development of the Parkersburg Water-Works might be permitted to point a moral, it would seem to be that of the advantage to be gained by every community in having a sovereign State organization, the duties of which should include investigations and experimental studies on all problems concerning water supply and sewage disposal. In this manner a large fund of valuable information might be collected, placing the supervising body in a position to advise local authorities in these matters in an impartial way and with freedom from the pressure of local controversial opinion. Fortunately, this has since been recognized in West Virginia, by the re-organization of the State Board of Health, and the addition to its staff of a competent sanitary engineer. Had the State had such an organization at the time of the adoption of the present water supply system, and had the City been obliged to have that Health Department approve the installation, the community might have been spared the expense of having many different engineers make separate investigations, only to disregard at the end the recommendations of most of them, and to put in a system which was largely experimental and in the adoption of which the theory and practice of water-works engineering was probably not the sole consideration.

Mr. Tolman.

EDWARD MAYO TOLMAN,* JUN. AM. SOC. C. E.† (by letter).—This comprehensive and valuable paper, together with the discussion on the so-called Smith system of filtration which it has evoked, has been of special interest to the writer, as that system has been made the subject of several bitter political controversies, into which one side or the other has endeavored to drag him, ever since his appointment as Engineer for the State Health Department, with demands that he either praise the system to the skies, or condemn it as utterly worthless. It was so evident that both sides, for the most part, had so far lost all sight of the engineering principles involved, that a careful report on the system would in no way change the opinion of those interested, but might serve as political capital, that the writer, heretofore, has refrained from an expression of opinion. Nevertheless, the Division of Sanitary Engineering of the State Department of Health of West Virginia has gathered considerable data on the system, some of which may prove of interest.

Beginning in January, 1917, the Department, once a week, has made complete chemical examinations of samples of the Ohio River from directly over the surface of the filters and from a tap in the City Hall. The results of five pairs of samples are given in Table 5.

These results would be of greater value if analyses of water from wells along the river bank were available for comparison. The series of samples were collected primarily for the purpose of showing the percentage of ground-water in the city supply, but the well samples showed such high contamination that the results are of no value in the present discussion. The State Chemist, Mr. Alton A. Cook, in commenting on these results, states:

"That practically no ground-water enters the supply is shown by the figures for ammonia, nitrates, hardness, and alkalinity, since typical pure ground-water would make a greater decrease in the figures for free ammonia than is shown here, and the analytical figures for nitrates, hardness, and alkalinity would tend to be considerably increased under these conditions."

This opinion is interesting, as it is generally thought that from 20 to 40% of the water collected in the Smith system at Parkersburg is ground-water. In the writer's opinion, the samples represent very special conditions which can occur only when there has been a rapid rise in the river following a considerable period of low stage, during which the ground-water has had a chance to run out. At Gallipolis, Ohio, and Owensboro, Ky., where somewhat similar systems have been adopted, it is claimed that they are filtering river water, not ground-water, and it may be that such is the case at Parkersburg. However, at Newell, W. Va., where the water supply is obtained from submerged

* Charleston, W. Va.

† Now Assoc. M. Am. Soc. C. E.

Mr. Tolman.

TABLE 5.—Results of Chemical Examinations of Water, Parkersburg Water-Works.

(Results, in parts per million.)

Date.	Jan. 24, '17.		Jan. 31, '17.		Feb. 7, '17.		Feb. 14, '17.		Feb. 22, '17.	
Source.	River.	Tap.	River.	Tap.	River.	Tap.	River.	Tap.	River.	Tap.
Color	slight.	none.	slight.	none.	slight.	none.	slight.	none.	slight.	none.
Turbidity	very high.	none.	high.	none.	high.	none.	high.	slight.	marked.	none.
Total ash	1055.	130.	180.	95.	320.	90.	75.	90.	190.	110.
Loss on ignition	320.	30.	120.	40.	110.	70.	130.	75.	50.	60.
Free ammonia	0.016	0.008	0.428	0.018	0.046	0.024	0.12	0.066	0.16	0.11
Alb. ammonia	0.91	0.05	0.14	0.04	0.33	0.05	0.13	0.036	0.11	0.04
Nitrites	0.013	0.002	0.010	0.002	0.012	0.002	0.022	0.010	0.014	0.008
Nitrates	0.23	0.31	0.24	0.23	0.39	0.14	0.14	0.28	0.22	0.15
Chlorides	11.5	12.4	10.6	13.8	13.8	12.4	17.5	16.5	25.8	20.7
Oxygen consumed	43.9	0.20	5.5	0.16	11.6	2.7	3.0	1.1	2.7	1.2
Total hardness	79.2	75.7	91.4	73.6	76.5	68.6	72.9	67.5	113.	108.
Alkalinity	20.5	28.	17.5	32.5	13.5	23.5	8.0	8.5	12.5	18.5
Iron	68.	trace.	6.8	0.05	13.2	0.02	2.3	0.	2.2	0.02
Sulphates	.....	.....	.....	.....	80.	62.	118.	98.	.....	.....

Mr. Tolman.

cribs, 4 ft. below the bed of the river, the contrary is certainly true, as shown by the three analyses in Table 6.

It is evident that, at this town, from 60 to 80% of the water is derived from the underground-water stratum, and is not filtered river water. Similar conditions obtain at Chester, Wellsburg, and other West Virginia towns along the Ohio River, which derive their water supplies from cribs or wells in the river bed. The underground waters along the Ohio in this State are very hard, and it is probable that sufficient underground water is entering the Parkersburg system to raise the hardness of that supply, during a large portion of the year, above that of a filtered river water, such as would be obtained by rapid sand filtration.

TABLE 6.—RESULTS OF CHEMICAL ANALYSES OF WATER, NEWELL WATER-WORKS.

(Results, in parts per million.)

Date.	MARCH 19TH, 1917.		
Source.	River.	Drilled well.	Tap.
Color	19.	0.	0.
Turbidity	220.	0.	0.
Total ash	180.	195.	185.
Loss on ignition	135.	90.	90.
Free ammonia	0.036	0.006	0.
Alb. ammonia	0.11	0.014	0.012
Nitrites	0.015	0.	0.002
Nitrates	0.18	0.70	1.04
Chlorides	11.8	10.6	6.9
Oxygen consumed	6.0	0.50	0.8
Total hardness	59.	175.	153.
Alkalinity	9.5	130.	95.
Iron	6.6	0.5	0.05
Sulphates	72.	98.	0.

Next, considering the character of the water from the standpoint of turbidity and color: No analyses of the tap water made at the State Laboratory have ever shown either color or turbidity. The highest turbidity recorded by Dr. L. O. Rose, of Parkersburg, who made daily tests throughout 1913 and frequent analyses since then, was 15. This followed a flood in the Miskindum River which empties into the Ohio a few miles above Parkersburg. At the time of the flood, Dr. Rose informs the writer, the results of ten very careful determinations of the turbidity of the Ohio at the filtration plant showed 25 000 parts per million, almost, if not quite, "straight mud." In the fall of 1916, the writer had one of his assistant engineers, Mr. Ellis S. Tisdale, visit some 200 residences and places of business in Parkersburg and inquire whether any turbidity had ever been noticed in the city water and whether the water had always been satisfactory from all other stand-

points. Not a single person interviewed had anything but praise for the supply. Mr. Tolman.

That the filtered water has not been highly turbid at times is remarkable, when some of the details of construction of the plant and of operation are considered. With the present design there is no method of telling what proportion of the water is passing through each filter unit. This is particularly dangerous, when it is considered that the pumps suck the water from the collecting manifolds. Thus, if the cover over any one filter should happen to be very thin, this unit would be operated at too great a rate. The water should flow by gravity from the collecting system to the pump-well, and each filter unit should be provided with a rate controller or a recording Venturi. Such devices would furnish data for the intelligent control of the filters. As it is, back-flushing is now conducted with no rhyme or rhythm, as can be seen clearly from an examination of Table 7, which shows a range of from only 1 300 000 gal. pumped per flushing to more than 92 000 000 gal.

TABLE 7.

Month.	1914.		1915.		1916.	
	No. of flushings.	Gallons pumped.	No. of flushings.	Gallons pumped.	No. of flushings.	Gallons pumped.
January	10	103 280 000	10	113 030 000	35	122 830 000
February	25	118 180 000	30	100 600 000	75	97 560 000
March	30	95 690 000	30	86 910 000*	25	87 350 000
April	0	92 460 000	45	88 830 000	40	113 710 000
May	10	95 350 000	60	115 880 000	20	97 970 000
June	35	115 170 000	10	99 760 000*	5	92 520 000
July	20	119 600 000	15	98 760 000	95	125 570 000
August	10	105 690 000	40	126 800 000	60	104 300 000
September	20	140 830 000	75	101 500 000	47†	114 980 000*
October	5	91 940 000	10	108 690 000	25	97 880 000
November	10	93 770 000	15	87 610 000	10	92 540 000
December	15	104 560 000	15	91 500 000	70	115 220 000

* Meter stopped.

† Filters cleaned.

The frequency with which the filters are flushed at present depends in no way on the quantity of water pumped or the character of the water. It is the writer's opinion that, in the absence of any controlling devices on the units, daily tests of the turbidity of the river should be made, which, combined with the stage of the river, whether rising or falling, and the quantity of water pumped, would probably serve as a basis for more intelligent filter operation. As it costs approximately $5.80 per million gallons to pump the water, and as between 40 000 and 50 000 gal. are used in a single flush, it will be seen that back-flushing has cost as much as $100 per month, a figure which could be reduced materially if more thought were given to the problem.

Mr. Tolman.

As to the purity of the water obtained by the so-called Smith system, at Parkersburg: The analyses made by Dr. Rose, up to the spring of 1916, showed a bacterial efficiency ranging from 95 to 100%, with an average of about 98.8 per cent. In the spring of 1916, the analyses made each month by the State Hygienic Laboratory began to show high bacterial counts and the presence of the colon organism in the filtered water. Dr. Rose then started making check analyses, and obtained similar, though less pronounced, results. To correct this condition, the city, in June, 1916, erected, in the pump-house, a Wallace and Tiernan chlorinating plant. The public has never been advised of this fact for fear that all manner of tastes and odors would be detected immediately. The results of the bacteriological analyses have been more satisfactory since chlorination was begun, but the count is still higher than it should be, and the colon organism is frequently encountered in 10-cu. cm. samples. It is possible that the *coli* found are entering at the reservoir. The water in the reservoir, owing to the location of the inlet and outlet pipes, has very little circulation, and probably some of it has stood there for many months. Dr. Rose informs the writer that samples taken at a depth have always shown a marked colon content.

In commenting on the results of the chemical analyses (Table 5), Mr. Cook states:

"In general, the purification is not all that can be desired, as shown by the figures for ammonia (free), nitrates, and oxygen consumed in one or more of the last three samples. The city drinking water, on the whole, is of fair quality, but there is room for improvement, before it can be considered a first-class drinking water."

It may be said, however, that Parkersburg has a satisfactory and safe water supply which is being delivered to the consumer at very low cost.

The writer has frequently heard the so-called Smith system of filtration condemned, on the ground that it cannot produce a safe or satisfactory water. This, he feels, is a mistake. At Owensboro, Ky., Gallipolis, Ohio, Chester and Newell, W. Va., and at other places using systems in many ways similar to that at Parkersburg, the water is satisfactory, both from chemical and bacteriological standpoints. At Chester and Newell, the colon organism has never been encountered, to the writer's knowledge, and yet no disinfectant is used.

It is the writer's opinion that the Smith system of filtration can furnish a satisfactory water at a low operating cost, provided nothing happens to injure the under-drains in the river, the problems of proper back-flushing being largely those of design and operation. It would appear then that, other things being equal, the choice of the Smith system over a rapid sand filter hinges on a gambling chance, the engineer betting that nothing will happen to the collecting drains against

the high cost of chemicals. If he wins, the profit to the city is large, but, if he loses, the city may pay in life, as well as in money. Mr. Tolman.

An interesting example of what may happen is afforded by experience at Wellsburg, W. Va., in 1916. The water supply of this town is derived from wells in the Ohio River, about 100 ft. from the shore. These wells are constructed of 60-in. cast-iron pipe set vertically in the river bed, with their upper ends 3 ft. below the surface of the sand. The bottoms of the pipes were left open, but the tops were sealed with iron caps leaded into the bells of the pipe.

The main suction line from the wells to the pump was provided with four branches, each branch extending through the cap of one of the wells. This system had been in use for a number of years, and had furnished a satisfactory drinking water. In the spring of 1916, however, one of the wells began to settle, which caused a crack in the suction main, and raw water entered the system. A typhoid outbreak of several cases and one death followed shortly.

When the supply was first used, it was possible to go back and forth in rubber boots over the surface of the sand-bar in which the wells were placed, consequently, the cost of the original construction was comparatively small. Since then, however, the Federal Government has built a dam across the Ohio just below Wellsburg, increasing the depth of the water over the bar by several feet. Investigation has shown that the cost of repairs to the well system would be almost prohibitive, and the town contemplates abandoning the present supply entirely and looking for a new source.

That a similar accident might happen to the Parkersburg system is shown clearly by the finding last fall (1916) of the broken strainer pipe which nearly caused a blow-out in the sand bed.

However, it must be stated that considering the lack of skilled attention given to this plant since it was put in commission, it has shown remarkable results. The plant is virtually in charge of the man at the pump, who, although very able, does not have the intimate engineering knowledge necessary for the proper operation of a practically untried form of filtration. The Water Commissioners are not trained engineers, and their supervision extends primarily to the financial side of the water-works problem. Parkersburg has adopted a very short-sighted policy in not placing a trained man in charge of the operation of the plant, and the fruits of this policy are now being noticed. It is probable that, if the plant had been operated scientifically, instead of in the hit-or-miss fashion of the past, the results would have been more to the credit of the system.

WILLIAM M. HALL,* M. AM. SOC. C. E. (by letter).—The writer is gratified with the extent of the discussion, and wishes to express his appreciation to each of those who have contributed to it. Mr. Hall.

* Parkersburg, W. Va.

Mr. Hall. The preliminary investigations for a new water supply for Parkersburg were commenced by Senator Camden—at his own expense—as early as about 1903, and were continued from time to time until Mr. Knowles was engaged in 1908. During this period he conferred with the writer many times. As stated by Mr. Fuller, two factions of the citizens were developed, one led by Senator Camden favoring a well supply, and the other led by the mayor of the city at that time, favoring a mechanical filtration system. Senator Camden spent several thousand dollars of his personal funds in extending the information as to the available natural supply obtainable from wells, and, besides, devoted much of his time to gathering information, locally and in the Ohio Valley, relative to the subject. He also presented the city with a tract of 43 acres for water-works purposes, but this was lost by the failure to adopt the well system.

The mayor in office at that period was skillful enough to use the subject so as to obtain two elections therefrom, but, on the third, it resulted not only in his complete downfall, but also in changing the three-headed form of city government—consisting of a council, board of affairs, and mayor—to one of simply a mayor and four commissioners working as a single body.

Mr. Johnson and others who wish for more data on the investigations by Mr. Knowles and Messrs. Fuertes and Fuller, in reference to the natural available well supply on the Camden Farm, the capacity of the wells, the comparison of the several waters chemically, the bacteria, *B. coli,* and bacteria count, should obtain printed copies of these reports from the city clerk; they contain many valuable data, too extensive for this paper.

A copy of the test made under the direction of Edward Mayo Tolman, Jun. Am. Soc. C. E., Chief Engineer of the State Board of Health, for the fiscal year past, as prepared by him and given out for publication under date of August 13th, 1917, is given in Table 8.

In a further report, in reference to this supply, Mr. Tolman says:

"The Parkersburg water supply is undoubtedly safe for consumption. * * * A few samples have been pronounced unsafe, and a few more suspicious. Some of the unsafe samples are undoubtedly due to the length of time elapsing between their collection and analysis. To be truly representative of the water, an analysis should be made within a very few hours after the sample has been collected, and yet at least one set of samples was 5 days in reaching the laboratory—ample time for the bacteria present to develop in untold numbers.

"As for the poor samples that cannot be accounted for in this manner, you must realize that Parkersburg has practically, if not actually, the only filter plant in the State that is not using large amounts of chemicals to obtain a satisfactory water. The addition of these chemicals is a large factor in reducing the number of bacteria. If Parkersburg will slightly increase the amount of chlorine now used,

Mr. Hall.

TABLE 8.—TEST OF QUALITY OF PARKERSBURG WATER SUPPLY.

Date.	River.			Pump House.			Reservoir.			Tap.			Interpretation is for tap samples only.
	Total count.	*B. coli*: 1 cc.	10 cc.	Total count.	*B. coli*: 1 cc.	10 cc.	Total count.	*B. coli*: 1 cc.	10 cc.	Total count.	*B. coli*: 1 cc.	10 cc.	
JULY–DECEMBER, 1916.													
7/ 7/16				8	0	+	0	0	0				
7/14/16							160	0	+	12	0	+	?
7/31/16				12 000	+	+	4 000	+	+	8	0	+	?
8/10/16				0	0	+	20	0	+	16 000	+	+	U*
9/ 2/16	No samples of river water taken during this period.									15	0	+	?
10/ 7/16							1 200	0	+	1 500	0	+	?
11/ 8/16				300	0	0	600	0	0	950	0	+	?
12/ 1/16				400	0	0	400	0	0	3 000	0	+	?
				0	0	0	0	0	0	120	0	0	S
										0	0	0	S
JANUARY–JULY, 1917.													
1/ 5/17	800	+	+	50	0	+	12	0	0	30	0	0	S
1/31/17	150	+	+	3	0	0	4	0	0	7	0	0	S
3/ 7/17	Bottle broken.			15	0	0	3	0	0	Bottle broken.			‡
4/ 2/17	2 000	+	+	1 500	0	0	2 000	0	0	1 000	0	+	?
4/28/17	400	+	+	2	0	0	6	0	0	5	0	0	S
5/31/17	500	+	+	2	0	0	250	0	0	3	0	0	S
6/30/17	2 500	+	+	2 500	0	0	4	0	0	1–2 500	0	0	S
										2–1 200	+	+	U†

Explanation: + Signifies presence of *coli*. 0 Signifies absence of *coli*. * 5 days in shipment. Results of doubtful value.
† Tap probably safe. No. 2 may have been accidentally contaminated in collection of sample. U Unsafe. ? Suspicious.
S Safe. ‡ Tap undoubtedly safe.

Mr. Hall. the bacteria contents of her drinking water can be made as low as that in any supply in the State. I have instructed one of my assistants to tabulate the results of analyses, and I am enclosing these data herewith.

"You will note that pump house samples have but seldom shown *coli,* one of these times, 8/10/16, evidently being a fluke, as the *colon* was found, yet no bacteria were present; a second was found in a set of samples that was five days in transit, and the third and only other time in a set of samples in which both the tap and reservoir showed up remarkably well."

In consideration of such a showing as to quality and cost, and the further fact that Mr. Gray, who passed on the final design of the infiltration system in use, has a professional experience of such wide extent as to be equalled by very few of his professional brethren, it appears that Mr. Fuertes's strictures on page 824 might have been best withheld until the Parkersburg system is more thoroughly tried out.

The writer omitted descriptions of the filters and their operations for the reason that they could be given better by Mr. Gray or his assistant, Mr. Leland. As they have not done this, the writer will answer briefly some of the queries suggested in reference to the general design.

Referring to the plan of the filter system (Fig. 14): Each of the five units in the system is connected independently with the pumps and flushing tank by an 18-in. pipe, with valves for shutting off one or more units from the remainder of the system, for back-flushing or cleaning the filters. At first, 12-in. pipes were proposed, but 18-in. pipes were adopted, in order to reduce friction and vacuum, and to regulate the supply better when flushing.

The design first considered for the pump, well, and pipes provided that the strainer pipes should flow into the well by gravity, which required a well some 20 ft. deeper than built. The writer still questions the advisability of the change in the plan. The principal objection to the arrangement as constructed is the vacuum produced in the suction pipes and the resulting injury to the filter beds by the great increase in the draft on them.

In flushing through the 18-in. pipes, the flushing water can be delivered to the strainer pipes with any desired rapidity or pressure. It is true that the water issuing from the strainers will take the line of least resistance in rising to the surface, and it is almost surprising that so few blow-holes have been found. However, the writer realizes that they might become a serious source of contamination.

The best method of removing the sand and gravel covering the strainer pipes, the extent of such needed renewals, the necessity of

repairing the pipes, and other requirements of operation which will occur as the system grows older, are yet to be developed. However, it is now apparent that extensive work in sand renewal or cleaning will only be required at intervals of several years. When such examination is made, should it be found that the polluted sand and gravel extends all the way down to the strainer pipes and that a coffer-dam is required for such renewal, even in that case, the cost should hardly equal half the cost of first construction. Should it be found that it is necessary to renew only the top sand, and that the work can be done in the wet, without a coffer for unwatering, it will cost only a small fraction of the first cost. Since the filters were constructed, the Government has built a dam in the Ohio River below them, which at pool stage puts about 11 ft. of water on them. Mr. Hall.

The writer is of the opinion that the City should be advised at all times of the extent and depth of the pollution, both in order to maintain the water at a higher standard of quality and also to be able to renew the affected sand and gravel before the pollution approaches so close to the strainer pipes as to affect seriously the quality of the water or make a coffer for cleaning imperative. However, thus far, the system has been operated without any such knowledge or any attempt to obtain it. In fact, except for the back-flushing and the 3 or 4 days' work done by Mr. Leland in the fall of 1916, no work has been done on the filters since their construction, not even to the extent of taking soundings in order to discover blow-holes.

From Mr. Tolman's commendation of this system, as previously cited, it is not clear that the State Board of Health would take such action as suggested by Mr. Knowles. In fact, it appears that any city having similar favorable conditions for such a water supply would have no serious trouble in obtaining the State Board's approval for a similar plant.

It may also be well to remember that the State Board is a creature of the State and the people, and this is a day when the autocrat of state, religion, and science seems to be passing.

When this system was adopted, the writer thought the conditions for a well system were so favorable that it was unwise for the City to take the hazard involved in the experimental system adopted; but his associates were so much in favor of doing so, on Mr. Gray's recommendation, that the writer finally concurred with them. The fact that the system in use has its source in the same underground supply as the proposed wells would appear to be more of a commendation than otherwise. After further study, he thinks that Mr. Knowles will conclude that there is a great difference between it and the old failing crib system tried by so many Ohio Valley towns.

On page 825 Mr. Fuertes makes a comparison of the cost of the mechanical filter system proposed by himself and Mr. Fuller, with

Mr. Hall. a statement of cost of the new construction as actually made. The details of their estimate, as taken from their report, is as follows:

Intake and intake well	$4 000
Filtration plant complete	50 800
Low-lift pumping equipment	3 000
Main pumping station, including all machinery except low-lift pumps	51 000
Dike and grading	10 000
Force main to reservoir	25 500
	$144 300
Add 15% for contingencies and supervision	21 645
	$165 945

A comparison of this estimate with the statement of the completed work on pages 780 and 781, including many things not in their estimate, is hardly relevant.

The following are the only items in their estimate which are of importance in making a comparison of the filters:

Intake	$4 000
Filtration plant	50 800
Low-lift pumps	3 000
Dike and grading	10 000
	$67 800
15 per cent	10 170
	$77 970

These are the figures the Commission used in comparing the cost of the mechanical filter with the one included in the Smith contract. The contract price was less than the foregoing estimate. With some additions, made after the first letting, the final cost was $80 700. This included all the work in the river, and the suction pipes up to the pump well.

As stated in detail by Mr. Burgess (page 813), Mr. Fuertes estimated the cost of operating the mechanical system proposed by himself and Mr. Fuller, with the pumping station, at $24 000 per year.

After looking over the accounts of the actual operation of the system in use for the past two years, in connection with the entire department accounts, the writer feels satisfied that the cost of operating the pumping station, including all the costs of putting the water in the City Reservoir and service pipes, has been less than $12 000 per year. Therefore, in the period during which it has been in

operation, as compared with a mechanical filter, as estimated by Messrs. Fuller and Fuertes, it has effected a saving of not less than $65 000. Mr. Hall.

There are only five men in the service of the City operating the entire pump station, day and night. As given in detail on page 781, the cost of managing and operating the filters has been insignificant, as compared with the requirements of a mechanical system, and the effluent is unquestionably much more pleasing to the consumers than some of the water from mechanical systems.

Relative to this matter, the following figures and data obtained from the City Auditor's report of accounts for the fiscal year ending June 30th, 1917, are interesting and pertinent:

"No payment has ever been made for either the old water-works system or the new, for either construction, maintenance, or operation, except that obtained from bonds and for water rentals. The average rate collected for metered water during the fiscal year ending June 30th, 1917, was 7.33 cents per 100 cu. ft., or per 748 gal. During the year the quantity of water which passed through the pump station, as measured by a Venturi meter, was 170 000 000 cu. ft., or an average of 3 484 000 gal. per day.

"54 483 061 cu. ft. passed through the consumers' meters, for which the city received about $40 000.

"From consumers paying monthly rental it received about $20 000."

Therefore, it appears that a little less than half the water pumped produces all the revenue, and the remainder was used for sprinkling, fires, sewer flushings, leakage, or by some unknown loss or waste.

Notwithstanding this low rate and the apparently excessive waste and leakage, the water-works fund is in a healthy condition. The bonds for the old water-works were long since paid. There is in the sinking fund, for paying new bonds, more than $92 000, and there is a surplus in hand of more than $20 000.

The writer prepared this paper solely for the benefit of the Profession and the City of Parkersburg, and wishes again to express his appreciation to those who have discussed it, and especially to Mr. Burgess for the data furnished, and to Mr. Potter for his discussion of the *Bacillus coli* and other remarks.

It now appears that the Parkersburg filters will probably be a success; but should the future develop any surprises, there may be a question as to whether such results are due to an incurable defect in design, or otherwise.

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Paper No. 1396

MULTIPLE-ARCH DAMS ON RUSH CREEK, CALIFORNIA*

By L. R. Jorgensen, M. Am. Soc. C. E.

With Discussion by Messrs. F. O. Blackwell, A. D. Flinn, F. W. Scheidenhelm, Edward Wegmann, Walter J. Douglas, Edwin Duryea, L. H. Nishkian, Gardner S. Williams, George W. Howson, and L. R. Jorgensen.

Synopsis.

This paper describes the design and construction of two multiple-arch dams built during the summers of 1915 and 1916. The determination of span length is first discussed, followed by general rules for the selection of the angle subtended by the individual arches. It is pointed out how the arch slope affects the load distribution on the arches, and the stability of the buttresses. Stress diagrams for the arches and buttresses are given, most of the calculations being graphical. The increase in deformation of concrete under sustained load, and its effect on stresses due to temperature changes is brought out, and an attempt is made to show that the time factor (time interval between the occurrence of maximum and minimum temperature) can to some extent be depended on to prevent or minimize temperature cracks. To stiffen the buttresses, counterforts are used in connection with struts. Test results of the concrete as used are given. Compression specimens were made in the field, and aged there, not under

* Presented at the meeting of April 18th, 1917.

laboratory conditions, but out in the open dry atmosphere. The up-stream faces of the arches were covered with a 1:2 coat of cement and sand, applied with a cement gun. In conclusion, some cost data are presented.

Introduction.

During the last few years several multiple-arch dams have been built in the United States, and, thus far, have been very successful in operation. Undoubtedly, many more would have been constructed had a thorough knowledge of their design and construction been more general among dam-building engineers. There are places where rock or earth fill dams, or a combination of the two, are now built, where multiple-arch dams could have been constructed more economically, and they would have been more substantial.

Although, under ordinary conditions, a rock or earth fill dam can be constructed with a sufficient, although unknown, factor of safety, such dams are absolutely unsafe under abnormal conditions, such as when water accidentally passes over the crest. It is a well-known fact that nearly all failures have been due to this cause. Water passing over the crest of a multiple-arch dam would not destroy it, and, for a more or less limited time, would not hurt the foundation, if this was otherwise at all safe for such a structure. A multiple-arch dam requires a good foundation, as the load is concentrated on the buttresses, and settlement of these would be likely to cause the collapse of adjoining arches. Whenever the foundation is solid rock, however, a multiple-arch dam can be constructed and will be as substantial as any, and more substantial than most types. The stresses and dimensions can be calculated with great accuracy. The factor of safety of such a structure, therefore, is known within narrow limits, assuming first-class construction, and on that account, precedents should not be given so much consideration as with rock and earth fill dams. These latter cannot be subjected to calculation of stresses, and therefore have to be built mostly along lines dictated by precedents. In general, it can be said that a multiple-arch dam of small and medium height (less than 100 ft. high) will cost less to construct than a rock fill dam, especially if the latter is provided with something better than a wooden up-stream face for the water-tight cut-off. Perhaps only in rare cases would there be occasion for comparing the relative costs of a multiple-

arch and a strictly earth fill dam, because, if there is enough suitable earth to construct an earth dam, a sufficiently good foundation for a multiple-arch dam and sufficient good building material for such a dam are not likely to be found at the same place, and *vice versa.*

For a multiple-arch dam higher than, say, 130 ft., the quantity of building material required, and therefore the cost of such a structure, increases quite rapidly, due mainly to the fact that the buttresses become very large and require more bracing. Eventually, therefore, a limit of height is reached, where it will be more economical to build a single arch across the canyon, unless the latter is very wide. This limit of height for any dam can only be found by trying all the types possible of application, as the shape of the dam site has also quite an influence on the design.

The Design of the Multiple-Arch Dam.

The first thing to be determined is the length of each individual span. Unless there should be strong reasons for using different span lengths for the several arches making up the complete structure, all spans should be the same, in order to facilitate the form work. Theoretically, the shorter the spans, the less the material required for the arches. The material required for the buttresses remains theoretically the same, no matter what length of span is chosen. Although a dam consisting of small spans takes less material than one where the spans are larger, the cost of construction may not necessarily be less. The form work becomes more extensive, and it is more difficult to place the concrete and reinforcing steel in the resulting narrow space between the form boards than in a wider space. Thin arch walls are more likely to collapse than thicker ones, and thin buttresses would require elaborate bracing in order to prevent their collapse long before their crushing strength had been reached. It is the arch that holds the water back, and therefore the water-tightness of the dam is to some extent a function of the thickness of this wall, although, to a much larger extent, it depends on the quality of the building material (concrete) used.

Taking all these facts into consideration, it may be stated that the practical and most economical span lies between the limits of 30 and 50 ft. For high dams, the economical span is near the upper limit; for low dams, it is near the lower limit. A 40-ft. span would be a

good average value for ordinary cases, and is chosen in the present instance.

The next feature to be determined is the length of the up-stream radius. It is known that the most economical arch* is the one that subtends an angle of 133½°, and that, for variations of about 10% on each side of this angle, the difference in economy is very small. For the dams to be described later the subtended central angle at the up-stream face is 120°, or, to be exact, 119° 57′. The volume of the arch has thereby been increased approximately 1% above the theoretical minimum, but, at the same time, the thickness has been increased 6%, thereby decreasing the ratio of thickness of arch to length of arch, which, for structural reasons, is a desirable feature, at least toward the crest where the thickness is small compared with the length. This also decreases the probability of percolation by decreasing the area of the wetted surface, and by increasing the thickness of the wall.

With the subtended angle (120°) and the span (40 ft.) decided, the length of the up-stream radius is calculated to be 23.1 ft. To facilitate form work, the length of this radius is kept constant from crest to foundation, except as noted later. Incidentally, this gives also the most economical arch, as the subtended angle is thereby kept practically constant. The arch is given a slope with the horizontal of 50°, in order that the water pressure, acting on the vertical projection of this slope, may tend to cut down the shearing stress on the buttresses to zero, or to some insignificant value.

The arch carries the total water load and a large part of the load due to its own weight. A preliminary arch thickness may be found by the simple formula,

$$P \times R_u = q \times t \qquad (1)$$

in which P equals the water pressure, in pounds per square foot; R_u equals the length of the up-stream radius, in feet; q equals the average stress, in pounds per square foot, of the area of the dam section under consideration; and t equals the thickness of the dam, in feet, at any given horizontal elevation.†

* Further information in reference to this may be found in *Transactions*, Am. Soc. C. E., Vol. LXXVIII, Paper No. 1322, pp. 689–690.

† The thickness of any horizontal arch slice (elliptical with major axis $= \frac{R_u}{\sin. 50°}$, and with minor axis $= R_u$) is made constant from abutment to abutment, and therefore the circular arch perpendicular to the slope must have its thickness increase from the crown toward the abutments.

After the thickness, t, has been determined, at as many points as thought necessary, say every 10 ft. apart in elevation, the weight of the arch can be calculated and the additional arch stress due to this load determined. If the total stress is found to be excessive, a new thickness will have to be chosen.

Due to the inclination of the arch, the water pressure is not distributed uniformly between the crown and the abutments on any imaginary arch slice perpendicular to the slope. As the crown is at a higher elevation, it sustains less water pressure than corresponding points at the abutments.

For the case under consideration—where the length of the upstream radius is 23.1 ft.; the subtended angle is 120°; and the inclination of the arch is 50° with the horizontal and 40° with the vertical—a point at the crown will always be

$$\frac{R_u}{2} \times \text{sin. } 40° = 11.55 \times 0.64379 = 7.424 \text{ ft.}$$

above a corresponding point at the springing line. Toward the crest this difference in elevation makes a large difference in the distribution of the load on the face of the arch, and therefore, also, in the location of the line of pressure in the arch ring. Take, for instance, an arch slice 1 ft. wide, with the middle of the crown at Elevation 5, and with the middle of this slice at the springing line at Elevation $5 + 7.424 =$ Elevation 12.424. With reservoir full to the crest, the water load at the middle point of the crown would correspond to $5 \times 62.5 = 312.5$ lb. per sq. ft., whereas the water pressure at the corresponding point at the springing line would correspond to $12.424 \times 62.5 = 776.5$ lb., or nearly 2.5 times more than at the crown.

At lower elevations this large difference (2.5 times) rapidly grows less, and becomes unimportant with crown elevations below Elevation 15. In the upper portion of the dam the radial component of the arch weight borne directly by the arch itself tends to neutralize the large difference in water pressure between the crown and the corresponding abutment points, as this weight component has its largest value at the crown, and its smallest value at the abutments. This can be shown most clearly by a graphical method, and in Figs. 1 and 2, two arch slices are drawn, each assumed to be 1 ft. wide. In Fig. 1 the middle of the crown is at Elevation 5, and then a width of 6 in. is taken on each side; and in Fig. 2 the crown is at Elevation 10. The

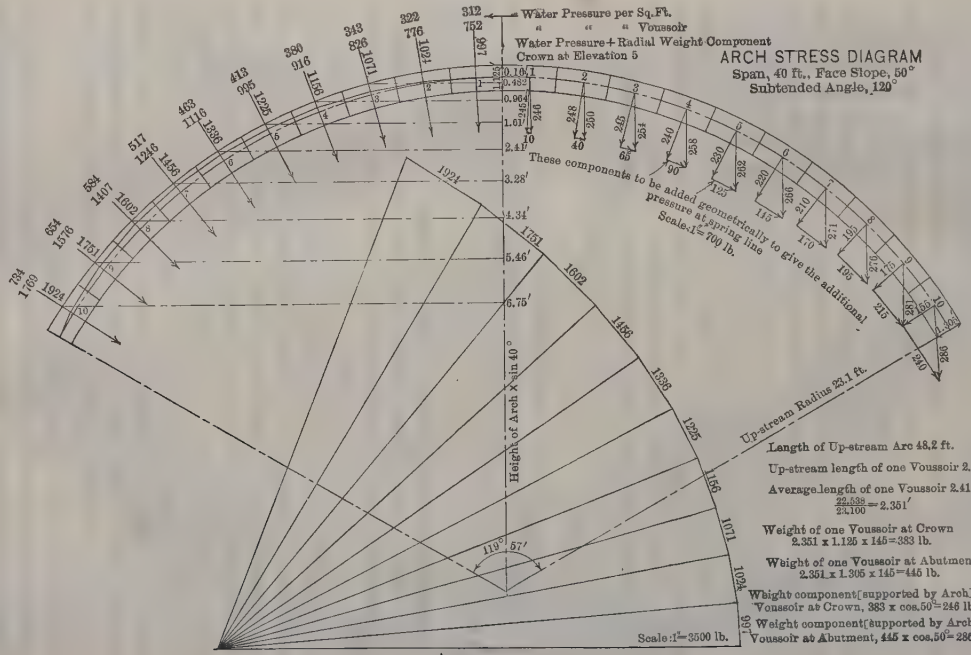

= Water Pressure per Sq. Ft.
" " " Voussoir
Water Pressure + Radial Weight Component
Crown at Elevation 5
ARCH STRESS DIAGRAM
Span, 40 ft., Face Slope, 50°
Subtended Angle, 120°
These components to be added geometrically to give the additional pressure at spring line
Scale 1"=700 lb.
Up-stream Radius 23.1 ft.
Height of Arch = 40'
119° 57'
Scale: 1"=3500 lb.
Length of Up-stream Arc 48.2 ft.
Up-stream length of one Voussoir 2.41 ft.
Average length of one Voussoir 2.41 x 22.538/24.100 = 2.351'
Weight of one Voussoir at Crown 2.351 x 1.125 x 145 = 383 lb.
Weight of one Voussoir at Abutment 2.351 x 1.305 x 145 = 445 lb.
Weight component [supported by Arch] of Voussoir at Crown, 383 x cos.50° = 246 lb.
Weight component [supported by Arch] of Voussoir at Abutment, 445 x cos.50° = 286 lb.

FIG. 1.

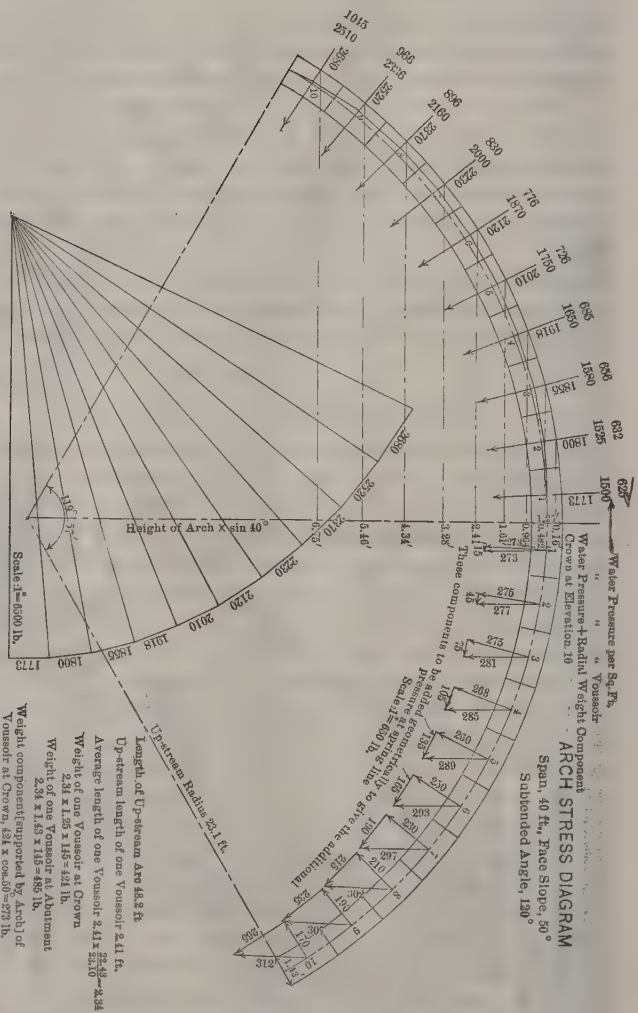

ARCH STRESS DIAGRAM
Span, 40 ft., Face Slope, 50°
Subtended Angle, 120°
Water Pressure per Sq. Ft.
" " " Voussoir
Water Pressure+Radial Weight Component
Crown at Elevation 10
These components to be added geometrically to give the additional Pressure at spring line
Scale: 1"=600 lb.
Height of Arch × sin 40°
Up-stream Radius 23.1 ft.
Scale: 1"=6500 lb.
Length of Up-stream Arc 48.2 ft.
Up-stream length of one Voussoir 2.41 ft.
Average length of one Voussoir 2.41 × 22.48/23.10 = 2.34 ft.
Weight of one Voussoir at Crown
2.34 × 1.25 × 145 = 424 lb.
Weight of one Voussoir at Abutment
2.34 × 1.43 × 145 = 485 lb.
Weight component [supported by Arch] of Voussoir at Crown, 424 × cos.50 = 273 lb.
Weight component [supported by Arch] of Voussoir at Abutment, 485 × cos.50 = 312 lb.

FIG. 2.

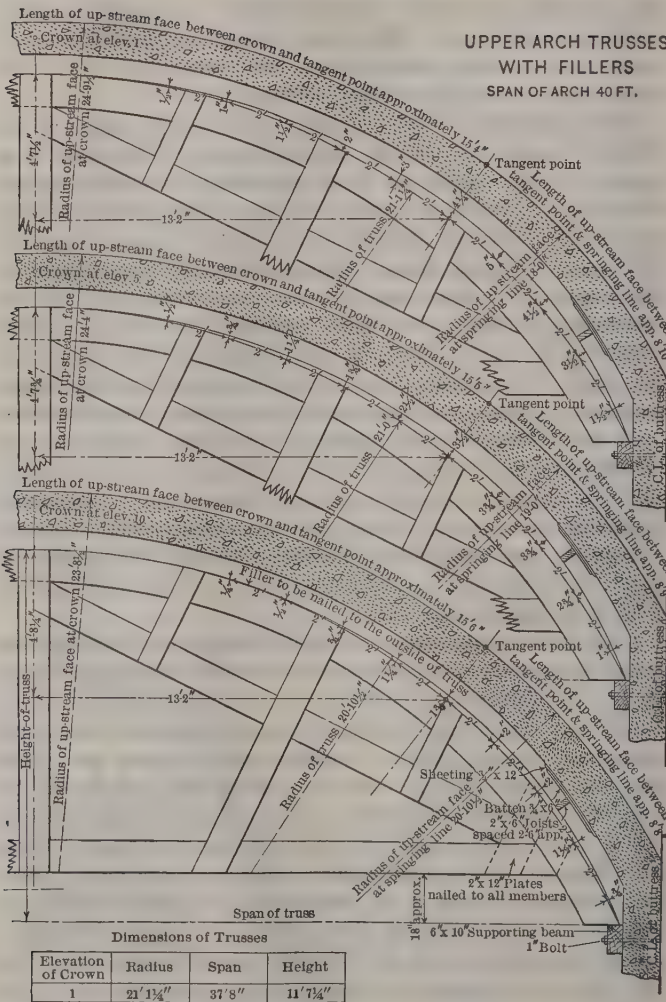
arch is divided into twenty voussoirs, and the forces acting on these voussoirs are calculated. These forces, though not distributed uniformly, are, of course, symmetrical on both halves of the arch, with respect to the center line between two buttresses.

On the right half of each diagram are shown the weights of the different voussoirs resolved into their components. The weight of a voussoir is partly transmitted to the base through the lower portion of the arch, and partly supported by the arch. The last component is equal to the total weight of the voussoir multiplied by cos. 50°, and is set off vertically, acting through the center of gravity of each voussoir. This vertical load is resolved into one radial component and one perpendicular thereto. The radial component can now be added directly* to the water load on the same voussoir, which, of course, is also radial. The geometrical sum of the last named (perpendicular) components represents the increased axial stress toward the abutments.

On the left half of each diagram the total radial forces acting on the different voussoirs are indicated. A polygon of forces is drawn, and, by using this, the line of pressure of the arch can be determined by drawing lines parallel to the proper rays in the polygon of forces. It is plainly seen that the line of pressure lies decidedly outside the center line of the arch; especially is this true of the arch with its crown at Elevation 5 (Fig. 1).

Using these diagrams, the correct shape of the wooden arch trusses supporting the form work for these upper elevations can be ascertained. The outside members of these trusses are made elliptical above Elevation 15, as shown in detail on Fig. 3, in such a way as to bring about coincidence between the line of pressure and the center line of the arch. At Elevation 15 the center line of the arch and the line of pressure do not exactly coincide, but they are close enough together to allow the circular shape of the arch to be used with entire safety. Below Elevation 15 the arch is made circular in a plane perpendicular to its sloping axis, and above Elevation 15 it is made elliptical in accordance with the diagrams, Figs. 1 and 2, or a slight modification thereof, depending on how far below the crest the maximum water level is to be. It might not be out of place to state here that J. S.

* For greater accuracy, this force should be multiplied by the ratio, $\frac{\text{length of mean radius}}{\text{length of up-stream radius}}$. It has been neglected in the present case, as this ratio is very close to unity.


UPPER ARCH TRUSSES
WITH FILLERS
SPAN OF ARCH 40 FT.
Length of up-stream face between crown and tangent point approximately 15′4″
Crown at elev. 1
Radius of up-stream face at crown 24′4½″
Tangent point
Length of up-stream face between tangent point & springing line appr. 8′10″
Radius of truss 21′1¼″
Radius of up-stream face at springing line 13′2″
Length of up-stream face between crown and tangent point approximately 15′6″
Crown at elev. 5
Radius of up-stream face at crown 24′4″
Radius of truss 21′0″
Length of up-stream face between crown and tangent point approximately 15′6″
Crown at elev. 10
Radius of up-stream face at crown 23′8½″
Filler to be nailed to the outside of truss
Height of truss
Radius of truss 20′10½″
Sheeting ¾″ x 12″
Batten 4″ x 6″
2″ x 6″ Joists spaced 2′0″ appr.
2″ x 12″ Plates nailed to all members
Span of truss
18″ approx.
6″ x 10″ Supporting beam
1″ Bolt
Lag bolts

Dimensions of Trusses

Elevation of Crown	Radius	Span	Height
1	21′1¼″	37′8″	11′7¼″
5	21′0″	37′6½″	11′7¼″
10	20′10½″	37′4½″	11′7¼″

FIG. 3.

Eastwood, M. Am. Soc. C. E., who has been very prominent in bringing into actual use the multiple-arch type of dam, builds the top portion of the arches vertical, whereby the circular shape can be used for the entire arch.

Stresses Due to Temperature Changes.

As dams are generally built during the summer season, it is only logical to assume that after their completion the individual arches are under tensile stress most of the time when the reservoir is empty, and decidedly so if it is empty during the cold season. The reinforcement in the arch, therefore, has been placed with the sole purpose of taking up these tensile stresses, which reach their maximum value near the down-stream face at the crown, and near the up-stream face at the abutments, under the conditions just stated. From the drawing at the left of Fig. 4 it will be seen that the reinforcing steel is placed at a distance equal to one-fourth of the total arch depth from the respective faces subject to tensile stresses, at the crown and at the abutments. The quantity of steel in the arch is perhaps not entirely sufficient to take care of the maximum condition of temperature drop, but it is believed that if tension cracks develop, the presence of the reinforcement will cause them to be minute and well distributed, and that, when the structure becomes loaded, the cracks will close tight. It was not deemed advisable to put in the arches more steel than that shown by Fig. 4, for the reason that it is of comparatively little use when the reservoir is full. It was also kept in mind that a large change in temperature is not likely to occur suddenly, as a time element of perhaps weeks or months is generally interposed between the occurrence of maximum and minimum temperature in a dam body. This time factor can be depended on, to some extent, to prevent or minimize temperature cracks. It gives the modulus of elasticity time to adjust itself to the new condition (colder or warmer concrete).*

The Stability.

After the buttresses have been given some preliminary dimensions, the stability of the whole structure can be investigated, most con-

* That the deformation of concrete increases, under sustained load, has been shown by tests and in practice. Two papers on the subject were read at the Twelfth Annual Convention of the American Concrete Institute, in Chicago, one by A. H. Fuller, M. Am. Soc. C. E., and C. C. More, Assoc. M. Am. Soc. C. E., entitled "Tests Showing Continued Deformation under Constant Load", and one by Mr. Carl B. Smith entitled, "The Flow of Concrete Under Restrained Load". This was abstracted in the *Engineering Record*, March 4th, 1916, p. 329. See also "The University of Minnesota Studies in Engineering, No. 3", by F. R. McMillan.

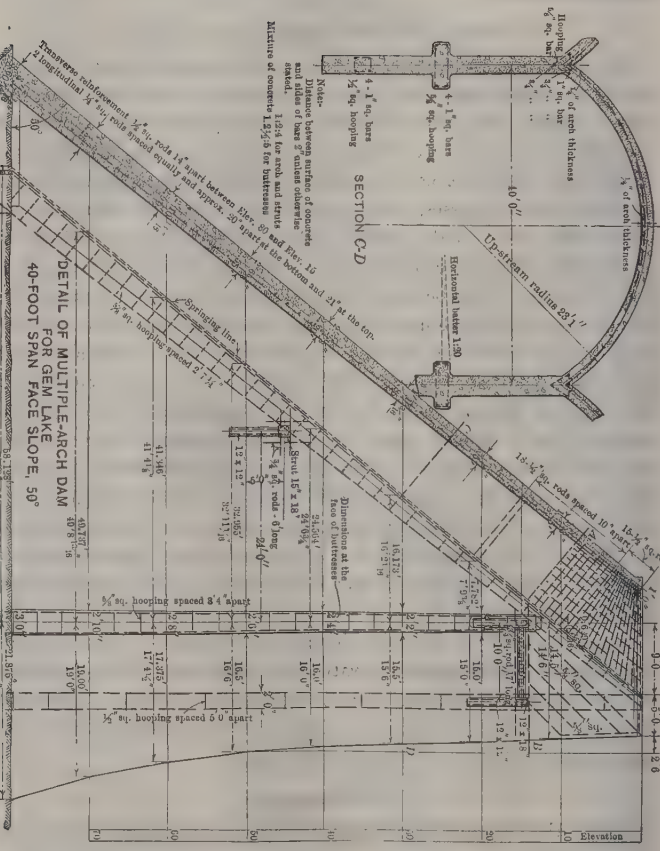

DETAIL OF MULTIPLE-ARCH DAM
FOR GEM LAKE
40-FOOT SPAN FACE SLOPE, 50°
SECTION C-D
Up-stream radius 23' 1"
Horizontal batter 1:20
Springing line
Elevation

FIG. 4.

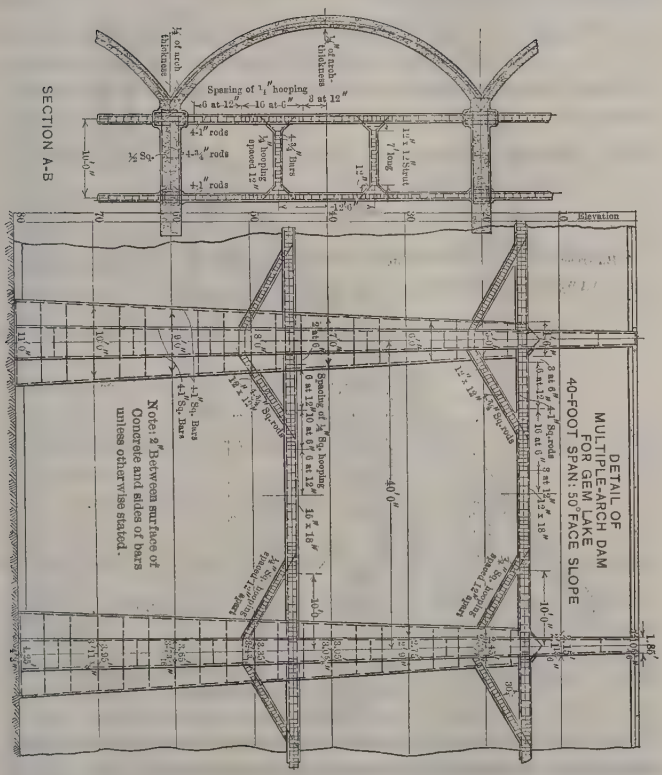

DETAIL OF
MULTIPLE-ARCH DAM
FOR GEM LAKE
40-FOOT SPAN: 50° FACE SLOPE
SECTION A-B
Elevation
Spacing of 1/4" hooping
6 at 12"
16 at 6"
3 at 12"
4-1" rods
4-3/4" rods
1/2 Sq.
1/4" hooping spaced 12"
12" x 12" Strut
7' long
Note: 2" Between surface of Concrete and sides of bars unless otherwise stated.
40'0"
1.85'

FIG. 5.

veniently by a graphical method, as shown by Fig. 6, representing a section through the crown of the arch. To facilitate the investigation, the dam is divided into horizontal sections, 10 ft. apart in elevation, and the forces acting on and above each section are shown to scale in the location and direction in which they act.

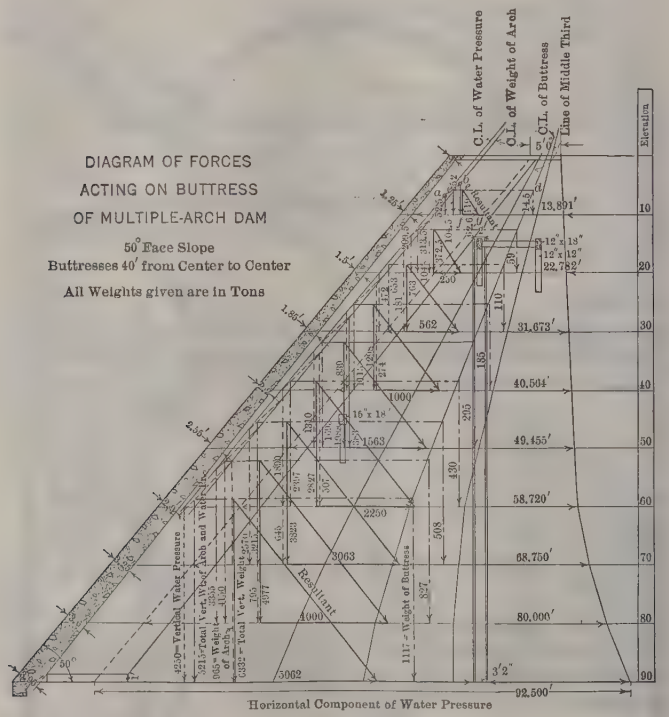


FIG. 6.

The most important force acting on the structure is the water pressure, and this, as usual, is assumed to be concentrated in a horizontal plane which is two-thirds of the total depth below the water surface.

The horizontal plane in which the water pressure is assumed to be concentrated intersects the up-stream face along an elliptical curve. The point of application of a single force, representing the water pressure on one total span of 40 ft., coincides with the center of gravity of

this ellipse, at least, as long as the water pressure does not penetrate the up-stream face skin, such as plastering, etc.

Due to the fact that the arches have been given a slope of 50° with the horizontal, the water pressure will have a vertical component $= \frac{\text{horizontal component}}{\tan. 50°}$. To a very great extent, the stability of the dam depends on the presence and action of this component, as it tends to hold the structure firmly down on its foundation.

The point of application of the water pressure, as already stated, is taken at the center of gravity of the ellipse. From the crown this is a little more than one-third of the total distance $\left(\frac{1}{3} + 5\%\right.$ for present conditions) between the crown and the springing line.

Considering first the upper 10 ft. of the dam, the horizontal plane in which the water pressure can be assumed to be concentrated is at a distance of $\frac{2}{3} \times 10$ from Elevation 0 downward, with the water surface at Elevation 0, and the point of application, a, of the single force is

$$\left(\frac{1}{3} + \frac{5}{100}\right) \frac{11.55}{\text{sin. } 50°} = 5.78 \text{ ft.}$$

from the crown, measured horizontally.

The horizontal water pressure due to the 10 ft. of water on the 40-ft. span is

$$\frac{0 + 625}{2} \times 10 \times 40 = 125\,000 \text{ lb., or } 62.5 \text{ tons.}$$

The vertical water pressure is equal to $\frac{62.5}{\tan. 50°} = 52.5$ tons. This latter force is now combined with the portion of the weight of the arch acting vertically. The weight of the arch above Elevation 10 is equal to its volume in cubic feet multiplied by the weight of 1 cu. ft. The center of gravity of the section—a trapezoid—is found, and the center of gravity, b, of the whole arch is then taken to coincide with the center of gravity of an ellipse through the center of gravity of the section, in the same manner as explained previously for the water pressure. The weight of the arch is now combined with the vertical component of the water pressure, and the location of their resultant is found by taking moments around either point, a or b. The numerical value of the resultant is equal to the sum of the two forces, $52 + 52.5 = 104.5$ tons. Taking moments around the point, a, and scaling the distances,

preferably on the sloping line between a and b for greater accuracy, we have the equation $\frac{52 \times 2.75}{104.5} = x = 1.367$ ft., giving the location of the resultant at a point 1.367 ft. from a along the line between a and b. The weight of the buttresses (above Elevation 10) assumed to be concentrated in the center of gravity, d, is calculated to be 14.5 tons; it is combined with the vertical load of 104.5 tons on the arch, in the same manner as shown previously, by taking moments around any point, say c. The location of the resultant is found to be at e, and its value is $104.5 + 14.5 = 119$ tons. This represents the total vertical force, both as to size and location.

This force is now combined with the horizontal water pressure of 62.5 tons, and the resultant is drawn. It is the purpose of this diagram, Fig. 6, to give the value of the resultant of all forces, and to establish the point of intersection, g, between this resultant and the base, in this case at Elevation 10. For convenience, the distance from e to f, representing the total vertical force of 119 tons, may be measured, and the distance, f-g, representing the 62.5 tons horizontal water pressure, may be set off on the base to the same scale.

Consider next the portion of the dam above Elevation 20 as a whole. The place of application of the concentrated water pressure is at a distance of $\frac{2}{3} \times 20$ ft. below Elevation 0, and the point of application of a single force representing the water pressure on the 40-ft. span is located the same as before, 5.78 ft. from the crown toward the springing line, measured horizontally. The vertical component of the water pressure $= \frac{\text{horizontal component}}{\text{tan. } 50°}$, acting on the arch between Elevations 0 and 20, is now combined with the weight of the arch lying between Elevations 0 and 20, and their resultant is drawn in the correct location, found by taking moments as shown previously. This resultant is again combined with the weight of the portion of the buttress lying between Elevations 0 and 20. The shape of the buttress is taken as that of an obelisk, and its volume and the location of the center of gravity are found from ordinary rules applying to such bodies. The weight of the struts and counterforts is to be added to the weight of the buttress, whereby the location of the center of gravity might be slightly changed.

The resultant of all vertical forces acting on the dam above Elevation 20 can now be found, both as to size and location, and combined with the horizontal water pressure, whereby the point of intersection of the resultant with the base at Elevation 20 is determined.

The same method of procedure is followed for the remaining portion of the dam, each time taking the base 10 ft. lower than in the preceding calculations, and the whole diagram, Fig. 6, is completed. If, now, the center line of the buttress, and also the two lines representing the middle thirds, are drawn, this diagram will point out very clearly whether or not the load is distributed economically on the buttress. For maximum economy, the resultant of all forces should intersect the base in the center line of the buttress; then the load will be distributed uniformly over the whole base. Toward the top this is not quite possible, and is not important, as the material there cannot be stressed very highly at any rate; but, toward lower elevations, the down-stream slope of the buttress should be shaped so as to conform with this condition, *viz.*, the resultant intersecting the base in the center line, or approximately in the center line, of the buttress. The total vertical load on the section shown in Fig. 6 is seen to be 6 332 tons, and the horizontal water pressure, 5 062 tons, both on a 40-ft. span. If the coefficient of friction is taken at 0.75, it is seen that the actual shear along the base amounts to only $5\,062 - 6\,332 \times 0.75 = 313$ tons. There is considerable steel in the section to help take up this shear, and therefore it was not deemed necessary to eliminate the shear entirely. There is no hydrostatic uplift to amount to anything acting on a dam of this type, and water could hardly find its way to lubricate the surfaces of possible cracks in the buttresses. Wherever it would be desirable to eliminate the shear entirely, the face slope should be made flatter, say 45, instead of 50 degrees. This, of course, adds to the material required for construction, but is the cheapest and best way of accomplishing the result.

Some difference of opinion may well exist as to the actual stress per square unit of area of the buttress at any horizontal elevation. The loads per buttress are given on Fig. 6. For instance, at Elevation 80, the vertical load is seen to be 4 977 tons, and the horizontal load, 4 000 tons. As the horizontal area is 360 sq. ft., the unit vertical stress should be $\frac{4\,977}{360} = 13.82$ tons per sq. ft., or 192 lb. per sq. in. The

shear would be a little more than friction alone would take care of, but steel is provided for the remainder. The resultant of the two forces (horizontal and vertical) intersects the base 2 ft. down stream relative to the center line, but on account of having the counterfort (20 sq. ft.) on this side of the center line also, the stress is actually distributed uniformly. Now, if, instead of the two principal forces, their resultant is used, assuming it as acting on a number of steps perpendicular to the direction of this force (the resultant), the apparent unit stress will be much higher. Thus, the resultant (6 385 tons) acting on an area equal to the sum of all the steps (360 $\times$ sin. 51° 13′ = 280 sq. ft.) will produce a compression equal to $\frac{6\,385}{280} = 22.8$ tons per sq. ft., or 317 lb. per sq. in.

There is a great difference between 192 and 317 lb. per sq. in. The actual unit compression will be somewhere between, undoubtedly depending on the relative value of the modulus of elasticity of the concrete for compression and for shear. It is seen that the first method, using the principal forces, gives the maximum possible shear that could occur, and that the other method gives the maximum possible unit compression that could occur. None of these methods is very satisfactory, as none fixes the absolute value of stress within narrow enough limits, but the writer knows of no better at present.

The reinforcing steel embedded in the buttress is put there for different purposes. Along the up-stream slope, the triangular steel construction shown in Figs. 4 and 5 ties the adjacent arches into the buttress. This is desirable on account of the fact that, in order to facilitate construction, the buttresses are built first, and the arches later. The hooping that interconnects the different bars is simply left protruding through the concrete of the buttress at the time this is built. Should one arch fail, this triangular girder would immediately take up the unbalanced thrust and prevent adjacent arches from collapsing, and this is its principal duty. Besides this, however, the steel is active in preventing cracks in the buttress and in taking up some shear. Toward the down-stream edge of the buttress, four vertical rods are embedded in the concrete for the purpose of stiffening it, preventing cracks, and taking care of wind pressure. Toward the middle portion, reinforced counterforts are constructed for the same purpose. These are still more effective in accomplishing this, due to

the greater distance between the pairs of steel rods. All material in the counterforts supports load the same as the buttress itself, but the counterforts, at the same time, are most effective in stiffening the large flat slab (buttress) so that fewer struts are required.

Toward the top of the buttress (at Elevation 15) two horizontal struts are tied to the vertical reinforcement (Figs. 4 and 5). These two struts are designed so that, besides their main purpose of holding the upper portion of the buttress in place, they are capable of supporting a light roadway. At Elevation 45 another strut is placed near the up-stream face, mainly in order to support the triangular girder, should the latter ever be required to support any unbalanced arch pressure. All struts are able to withstand tension as well as compression, as can be judged from an inspection of the details on Figs. 4 and 5.

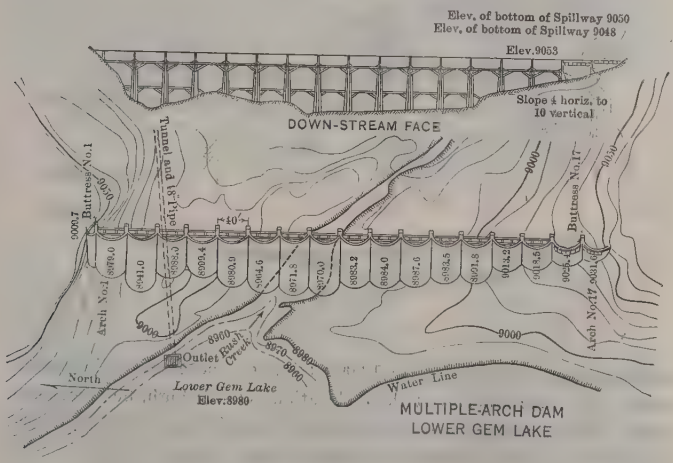


Fig. 7.

Gem and Agnew Lake Dams.

The design shown in the foregoing illustrations was made by the writer for the Pacific Power Corporation, of Bodie, Cal., to be used for the construction of their Gem and Agnew Lake Dams, on Rush Creek, Mono County, California. Fig. 7 is a plan of the Gem Lake Dam and dam site, and Fig. 8 shows the Agnew Lake dam site. On Fig. 7 is also indicated the outlet works, consisting of a concrete

chamber provided with a row of iron bars in front, and a 48-in. steel pipe laid through a tunnel for about 300 ft. This pipe line terminates in a power-house 1 808 ft. below (in elevation) the top contour of the lake. The maximum height of any individual arch of this dam is 84 ft., and the vertical distance from the deepest point in the foundation to the crest is 112 ft. The length across the dam site at the crest (Elevation 9 053) is 700 ft. The reservoir created has a capacity of 17 000 acre-ft., and is capable of regulating Rush Creek to yield an estimated average flow of 45 sec-ft. The drainage area above the dam is 22½ sq. miles, and is on the eastern slope of the Sierra Nevada Mountains. Its elevation ranges from 9 000 ft.

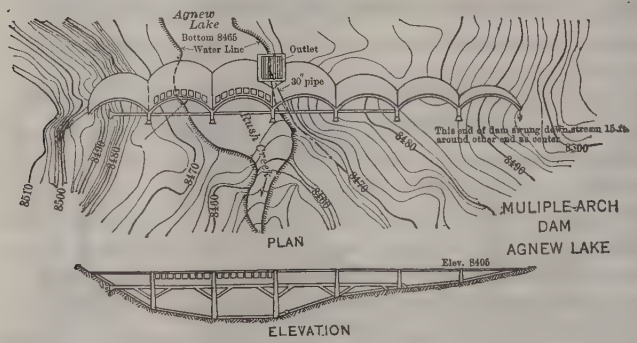
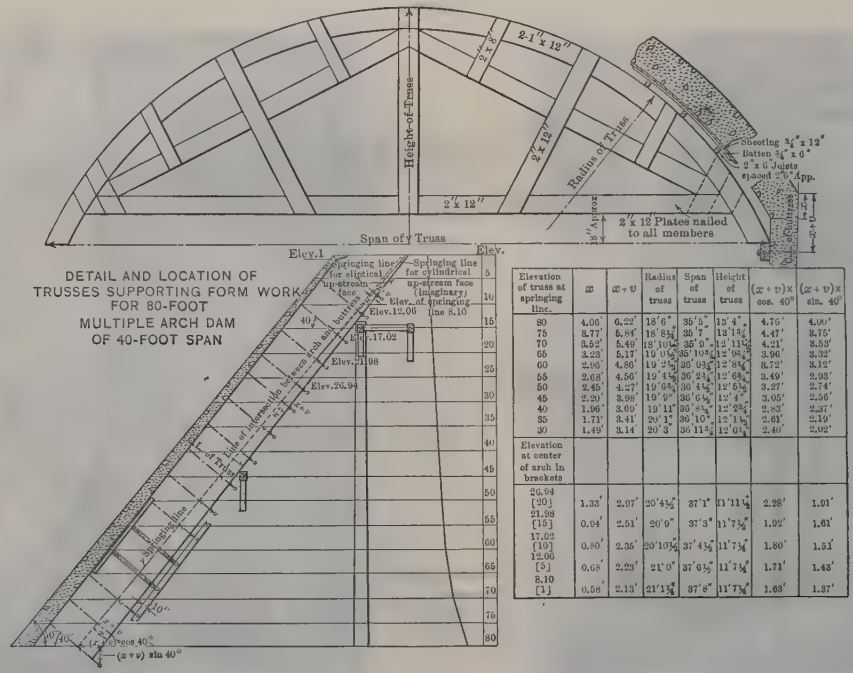


FIG. 8.

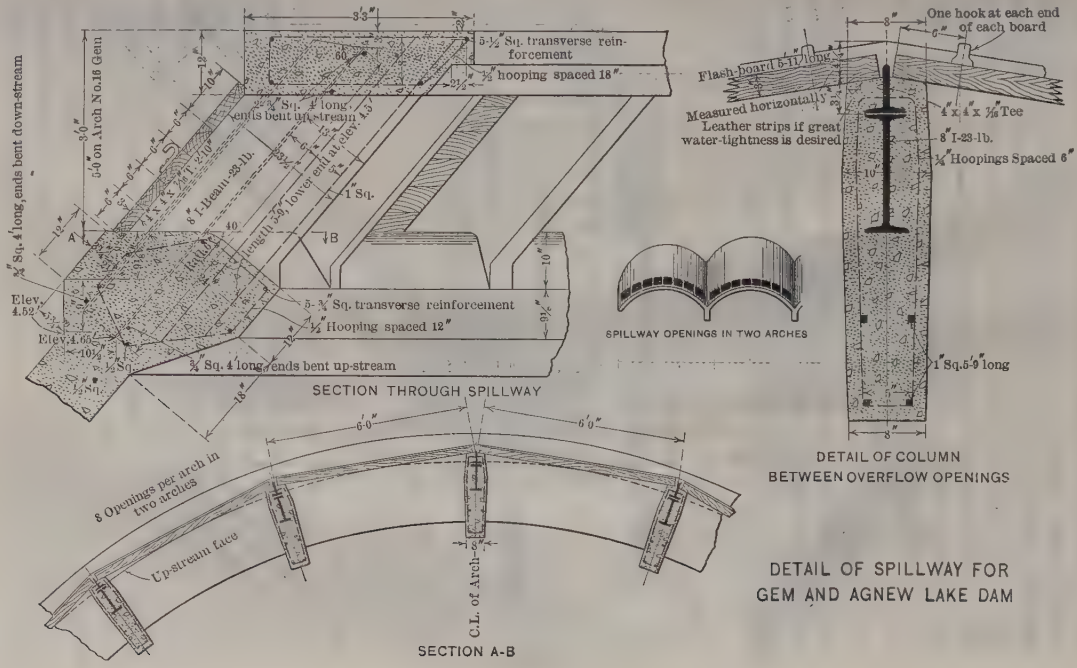
above sea level at the dam to 12 000 ft. along the crest of the mountains. The precipitation at these elevations is still high, but decreases very rapidly from the foot of the steep mountains out over the desert immediately east, at elevations ranging from 6 000 to 7 000 ft. above sea level.

The rock at the dam site, for the most part, was exposed bed-rock worn clean by glacial action. Some excavation, however, was necessary in the stream bed and through a rock slide at the north abutment. The building material for the dam was found near-by. The sand was taken from the shore of the natural lake. The rock had to be hauled a short distance on a tramway, first from the outlet tunnel dump (limestone), and later from a large rock slide (granite) about


2-1" x 12"
2" x 12"
Height of Truss
Radius of Truss
Sheeting ¾" x 12"
Batten ¾" x 6"
2" x 6" Joists
spaced 2'0" opp.
2" x 12" Plates nailed to all members
Span of Truss
Elev.
DETAIL AND LOCATION OF TRUSSES SUPPORTING FORM WORK FOR 80-FOOT MULTIPLE ARCH DAM OF 40-FOOT SPAN
Springing line for elliptical up-stream face
Springing line for cylindrical up-stream face (imaginary)
Elev. at springing line 8.10
Elev. 12.05
Elev. 17.02
Elev. 21.98
Elev. 26.94
Line of intersection between arch and buttress
Springing line
40'0"
(x + v) cos 40°
(x + v) sin 40°

Elevation of truss at springing line.	x	$x+v$	Radius of truss	Span of truss	Height of truss	$(x+v)\times$ cos. 40°	$(x+v)\times$ sin. 40°
80	4.06′	6.22′	18′6″	35′5″	13′4″	4.76′	4.00′
75	3.77′	5.86′	18′8¾″	35′7″	12′11½″	4.47′	3.75′
70	3.52′	5.49′	18′10¾″	35′9″	12′11½″	4.21′	3.53′
65	3.23′	5.17′	19′0¼″	35′10¾″	12′9⅞″	3.96′	3.32′
60	2.96′	4.86′	19′2½″	36′0¾″	12′8¾″	3.72′	3.12′
55	2.68′	4.56′	19′4¾″	36′2¾″	12′6¾″	3.49′	2.93′
50	2.45′	4.27′	19′6¾″	36′4⅝″	12′6¾″	3.27′	2.74′
45	2.20′	3.98′	19′9″	36′6⅝″	12′4″	3.05′	2.56′
40	1.96′	3.69′	19′11″	36′8⅝″	12′2¾″	2.83′	2.37′
35	1.71′	3.41′	20′1″	36′10⅝″	12′1¾″	2.61′	2.19′
30	1.49′	3.14′	20′3″	36′11⅝″	12′0¾″	2.40′	2.02′
Elevation at center of arch in brackets							
26.94 [20]	1.33′	2.97′	20′4½″	37′1″	11′11¾″	2.28′	1.91′
21.98 [15]	0.94′	2.51′	20′9″	37′3″	11′7⅛″	1.92′	1.61′
17.02 [10]	0.80′	2.35′	20′10¾″	37′4½″	11′7¾″	1.80′	1.51′
12.06 [5]	0.68′	2.29′	21′0″	37′6⅛″	11′7¾″	1.71′	1.43′
8.10 [1]	0.58′	2.13′	21′1½″	37′8″	11′7¾″	1.68′	1.37′

FIG. 9.



DETAIL OF SPILLWAY FOR GEM AND AGNEW LAKE DAM

Fig. 10.

FIG. 11.—UP-STREAM FACE OF AGNEW LAKE DAM, SHOWING SPILLWAY OPENINGS AND INLET.

FIG. 12.—DOWN-STREAM FACE OF AGNEW LAKE DAM, SHOWING SINGLE STRUT AND BUTTRESSES, AND ALSO COPING ALONG CREST OF ARCH.

Fig. 12.—Dunstanville Mountain, from Lake Okataina, showing Rua-tahuna Summit and Tarawera Cone, also Dome along Chasm of Tarawera.

FIG. 13.—GEM LAKE DAM.

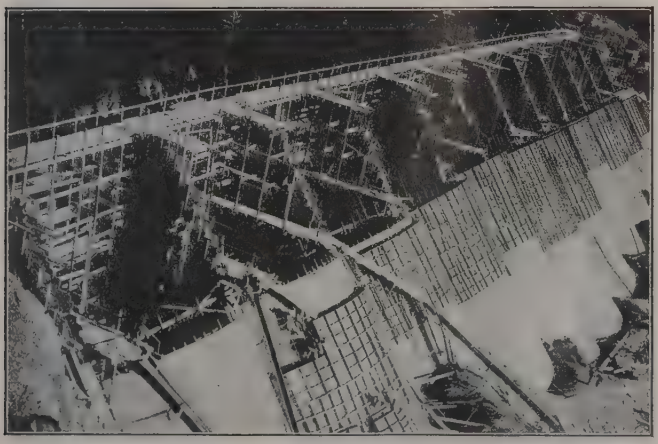


FIG. 14.—GEM LAKE DAM, SHOWING SECTION OF UP-STREAM FORM BEING HOISTED INTO PLACE, ALSO MAIN UPPER RUNWAY WITH BRANCHES ALONG BUTTRESSES.

FIG. 15.—GEM LAKE DAM, SHOWING PROGRESS OF FORM BUILDING.

FIG. 16.—APPLYING PLASTER TO UP-STREAM FACE WITH CEMENT GUN.

FIG. 17.—GEM LAKE DAM. WATER LEVEL 35 FT. ABOVE TUNNEL FLOOR.

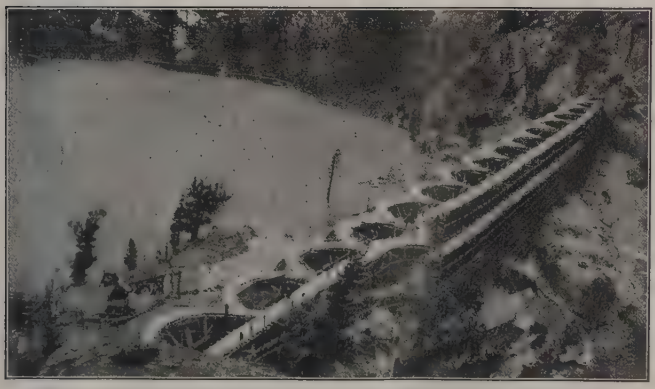


FIG. 18.—GEM LAKE DAM, SHOWING RUNWAYS AND CONCRETE ELEVATOR.

2 500 ft. away. All available materials in the neighborhood, especially the different sand deposits, were tested before any particular material was selected for construction. As the sand deposit along the shore of Gem Lake was good, it was used. This sand was first pumped, and later shoveled, from the lake, and transported to a storage pile near the mixing plant. This lake sand, which contained 3½% of clay and 1% of dirt, was mixed with the sand from the rock crusher (all particles being less than ¼ in. in diameter) in the proportion of about three-fourths of lake sand to one-fourth of crushed rock sand. This gave a very good combination, both as to strength and watertightness.

Compression tests on 6-in. cylinders were made as the work progressed, using Bear brand Portland cement, Gem Lake sand, and crushed rock, in the proportions 1:2:4, and averaged about 900 lb. per sq. in. for crushing at the age of 14 days.

A 1:2:4 mix was adopted for the arches and struts, and a 1:2½:5 mix for the buttresses. The actual proportions, however, were sometimes changed, but 1½ bbl. of cement for the arches, and 1¼ bbl. for the buttresses were used always. The rock was crushed in a gyratory crusher, and separated into three sizes through a revolving screen having 1½, ¾, and ¼-in. meshes. The rejects from the screen went into a jaw crusher, the jaws of which were set to give a maximum size of 2 in.

The distribution of the concrete to the different arches and buttresses was done with two-wheeled push carts and short chutes.

During the construction period, in the summers of 1915 and 1916, tension tests were made of cement briquettes in a small field laboratory. Much attention was also paid to the sand used, and 1:2 and 1:3 mortar briquettes were tested frequently; silt analyses were made as the work progressed.

The reinforcement placed in the dam consisted of high-carbon steel bars, either corrugated or twisted. As the position and details of these bars are fully shown in the figures, no further explanation need be given.

The trees standing on the reservoir site were cut down, sawed into lumber in a mill erected on the ground by the contractors, and used for the forms. Large cone-bearing trees can be found in this neighborhood up to Elevation 11 000 in fairly large quantities, but

not of the best quality. The lumber proved to be good enough for form work, was used green, and on that account ¾ by 4-in. battens were placed on the under side of the sloping arch to prevent the leakage of grout between the boards when shrinkage took place. These battens also helped to give an even, circular shape to the outer ¾ by 12-in. boards, constituting the form for the down-stream face. This sheeting was supported on 2 by 6-in. studs, 2 ft. 6 in. apart, toe-nailed to the wooden arch trusses shown in Figs. 3 and 9. These trusses were 5 ft. apart in elevation, and a little more than 6½ ft. apart, measured along the arch slope. The form work for the up-stream face consisted also of ¾ by 12-in. boards nailed to 2 by 6-in. studs, which in turn were nailed to circular boards at regular intervals. This outside form was held away from the inside form (down-stream side) by wooden distance pieces of the proper length, which pieces were removed just before the concrete reached them.

A 1:2 plaster coat of cement mortar ¼ in. thick at the crest, and increasing to ¾ in. thick 80 ft. below, was put on the up-stream face with a cement gun.

At the south abutment the two last arches are provided with spillway openings. These spillway openings can be closed with loose flash-boards. This it is proposed to do toward the end of the wet season, so as to fill the reservoir to within, say 1 ft. of the crest of the dam, thereby gaining 2 ft. of water over an area of nearly 300 acres. The spillway is shown in detail on Fig. 10.

Cost.

The Gem Lake Dam contains 8 537 cu. yd. of concrete and 82 tons of reinforcing steel. The contract price was $22 per cu. yd., including cement, forms, plastering the up-stream face, and all tools and materials except the reinforcing steel, which was paid for as an extra at the rate of $110 per ton in place. The excavation, of which there was only a limited quantity, was also paid for as an extra. The high cost is explained by the fact that freight rates were high, and, further, that the distance from the railroad to the power-house site at Silver Lake, at the foot of the steep mountains, was nearly 60 miles over desert roads with heavy grades. From the power-house site, a tramway, approximately 4 500 ft. long, took all supplies up the mountain side to Agnew Lake, 1 250 ft. above. At the outlet of this lake, the

Agnew Dam, a smaller structure, 30 ft. high and 280 ft. long, similar in design to the Gem Lake Dam, but having only one strut, was built at the same time and at the same unit prices, in order to increase the capacity of the natural lake. All material for use in the construction of the Gem Lake Dam was brought across this lake on a barge, for a distance of about 2 000 ft., to the foot of another tramway terminating at the site of the Gem Lake Dam, 550 ft. higher in elevation. The cost of these tramways was not included in the price per cubic yard for concrete, but was paid for as an extra. These tramways were also necessary for the construction of the pressure pipes to the power-house, one from Gem Lake, and one from Agnew Lake, and would have had to be built independent of the dams.

For the long haul across the desert from the Southern Pacific Railroad station at Benton, Cal., to the power-house at Silver Lake, six 75-h.p. C. L. Best tracklayers, burning distillate, were used, each hauling three trailers. The net load was as close to 20 tons as practicable, and the time necessary to make one round trip was about 6 days of 12 hours, including loading, unloading, and ordinary delays. The speed per hour of these tracklayers was 2½ miles on high gear, and 1½ miles on low gear. The contractors found that the cost to them for hauling this distance was at the rate of $13.50 per ton. The hauling of the cement and materials for the dams was not paid for extra, but included in the price for concrete in place.

A rock fill dam on the same site would have had to be built for $2.15 per cu. yd. (construction cost), including the water-tight face and hand-laid rock, in order to be on equal terms, as to cost, with the multiple-arch dam built. In the writer's opinion, this would not have been possible in this place. In any locality where cement can be laid down at less cost than in this case, the relative cost of a multiple-arch dam and a rock fill dam will be still more in favor of the multiple-arch dam.

The writer furnished the designs and also supervised the construction of the dams.

Mr. C. O. Poole was Chief Engineer for the whole development, E. J. Waugh, Assoc. M. Am. Soc. C. E., was Resident Engineer, L. B. Curtis, M. Am. Soc. C. E., Field Engineer, and Mr. F. O. Dolson, Superintendent of Construction.

Messrs. Duncanson Harrelson Company, of San Francisco, were the contractors for both dams.

DISCUSSION

Mr. Blackwell.

F. O. Blackwell,* M. Am. Soc. C. E.—This paper is a valuable contribution to the theory of multiple-arch dam design, particularly as to the economical spacing of the piers and the thickness of the arch. The dam sites are on small drainage areas, high in the mountains, where labor and materials are very expensive. It has been stated† that the cement for these dams cost $7.50 per bbl.

A very conservative design might have been prohibitive in cost, and probably the region is uninhabited and no damage to life or property would result in case of failure.

The sections appear to be very thin for a 40-ft. span, and the reinforcement is extremely light. The use of more concrete would have added much to the ability of the structure to resist unexpected floods, ice thrust, changes of temperature, etc.

It is always desirable, too, to have a good margin over theoretical quantities in order to allow for poor material and workmanship, which will occasionally be found, no matter how careful the engineeer may be.

A fair comparison between this dam and other types is difficult to make, as the standards are quite different, but the multiple-arch appears to be cheaper than any other kind under the conditions which existed at the location, except possibly a rock fill with a thin vertical core-wall. The author states that there was loose rock but no earth at the sites.

The usual type of solid, retaining-section, gravity dam would have required about four times as much concrete as was used in the dam as built, but from 25 to 40% of cheap rock might have been laid in thick concrete sections, and concrete containing less cement might have been used. The form work and cost of forms and placing would also be less in a large mass.

Including engineering and overhead expenses, the multiple-arch dam probably cost about $30 per cu. yd., so that a gravity dam would have to be built for $7.50 per cu. yd. in order to equal it in cost.

In an accessible location, and with concrete rubble masonry at from $3 to $4 per cu. yd., the solid dam, in the speaker's experience, has always figured out to be cheaper than any multiple-arch or reinforced design.

Compared with a reinforced concrete slab dam, the multiple-arch seems to have the advantage in cost. The piers would have to be twice as many, the deck thicker, and the reinforcing metal in the deck about ten times as great.

* New York City.

† In an article in *Engineering News*.

Mr. Blackwell.

One of the principal objections to a multiple-arch is the fact that the sections are all dependent on each other for stability. If one pier should settle, the arches on each side of it would break, and, as there is nothing to take the thrust of the remaining arches, they would push over their piers, and the whole dam would collapse. The struts between the piers could not be relied on to save the dam from complete failure, as the movement of any pier would throw them out of line and cause a resultant thrust, tending to push the pier in the direction in which it had already started to move. Rigid foundations, such as the author states exist in this case, are essential.

The Great Western Power Company started to build a multiple-arch dam at Great Meadows, on the Feather River, California, but abandoned it in favor of a rock fill with an earth face sluiced into place on the up-stream side to make it water-tight and cover the bottom for a considerable distance up stream. The lava foundations, on being opened up, were found to contain mud seams, which might have moved under the piers. As a general principle, a dam should be no better than its foundations. On a yielding bottom a dam of a flexible type is required.

There is one important matter that Mr. Jorgensen does not mention: At an altitude of 9 000 ft. in the Sierras it must be very cold at times, and thick ice must form in the reservoir. The thin arch is not much protection against cold, ice must form to a considerable thickness on the face of the dam, and such cracks as develop will be filled with ice. There may also be ice thrust against the dam, which could have serious results. With a mass of ice frozen fast to the arches, the inclination of the face would not relieve this pressure. Floating masses of ice may also drift against the arches, thus causing concentrated pressures on parts of the dam, and the arches are not designed to stand unbalanced pressures. One advantage of the ordinary form of masonry dam is the weight of its large mass, which will stand considerable impact without damage.

The spillway does not look very large, and would be likely to be choked with floating ice, in which case the water would go over the top of the arches. If this occurred, and the water carried ice with it, the ice would fall on the struts between the piers and might damage them. A deck on top of the dam would carry the water so that it would fall below the dam and not injure the struts and the foundations. It would also strengthen the top of the arch against ice thrust and would tie the piers together better.

A pressure of nearly 23 tons per sq. ft. seems to be extreme for a practically unreinforced and slender pier 90 ft. high.

The coefficient of friction against sliding on the foundations is 0.80 as compared with 0.65 in most solid dam designs. If the rock were smooth or horizontally stratified this might be unsafe.

Mr. Blackwell. It would be very interesting to hear from the author as to whether there has been any leakage through the arches, or whether any trouble has been experienced from ice.

Mr. Flinn. A. D. FLINN,* M. AM. SOC. C. E.—Minimizing the quantity of masonry to be used in a dam naturally occurs to any one who has given much thought to the design of large masonry dams. The idea of accomplishing this by arches and buttresses seems to have occurred to engineers in different parts of the world many years ago, for such dams have been built in India and Australia, as well as in America. They have sometimes been called buttressed dams. There appear to be three important varieties: One, of which the Meer Alum Dam in India is an example, has the arches vertical, with the buttresses sloping on their down-stream sides or faces; a second variety is similar, but has the spandrel spaces filled with masonry; and the third variety, to which the dams described in the paper belong, has the arches inclined, with the down-stream faces of the buttresses vertical, or nearly so.

Mr. Blackwell has referred to the disadvantages of thin sections in such structures. The speaker would like to emphasize that point, on the basis of some experiences with concrete within the last few years. Possibly, some other engineers have also had experiences showing that Portland cement concrete under certain conditions disintegrates when exposed to weather, and particularly when exposed to weather and water together. The cause for such disintegration is not yet fully known, but it seems to be due in part to abuse of the concrete (or rather of the cement in it) by the use of excessive quantities of water in mixing, and leaving it in the concrete when it is finally placed in the forms. The speaker knows of one or two cases in which disintegration, apparently due to this cause, has gone on rapidly. Therefore, it is quite evident that a structure like a dam, with walls as thin as 12 to 24 in., would not long resist such an attack, if it should set in.

Another question—one of economy—has occurred to the speaker. In the East, dam sites are usually overlaid with glacial drift or other earthy deposit to considerable depths. Rarely does one find the nearly exposed rock foundations with which the West is favored in many places. It would seem that digging through a considerable depth of earth for the arches and the buttresses would be a handicap to the multiple-arch dam, in comparison with the "solid" masonry dam.

Another thought which occurs to one in looking at dams of this design, is along a line on which American engineers have not directed very much attention until within the past 2 or 3 years, and that is, the vulnerability of structures to attack either by malicious persons or by an enemy. It seems to the speaker that a "hollow" dam might

* New York City.

be easily wrecked, if not guarded, in time of war, strike, or riot, and that a person of malicious intent, in the case of any such trouble, could readily do a structure of this kind great injury in a few hours, or even in a briefer time. One of the important considerations in providing for the guarding of a structure is to have it so arranged or so strong that it can be easily protected, and by the minimum number of troops. Mr. Flinn.

In comparing the three forms of multiple-arch dams which have been mentioned, the question naturally arises: What are the relative advantages between the arch set vertically with the buttresses sloping on the down-stream side, and the type of dam described in the paper, with the arches sloping and the buttresses having vertical faces on the down-stream side? Is there a distinct advantage in either case, or is the difference solely due to an endeavor to secure patent rights, or some other commercial advantage? If any variety of multiple-arch dam has advantages over the others, will not the author or some other engineer who has devoted special study to this type, state these advantages? The speaker is interested as to the readiness or difficulty of making a dam of this kind, together with its foundations, sufficiently water-tight, and what standard of water-tightness is accepted. Would the leakage permitted be greater than that from a well-built "solid" masonry dam? As time goes by, do the agencies which make for greater water-tightness prevail over those which tend to produce leaks?

F. W. Scheidenhelm,* M. Am. Soc. C. E.—The author has presented an interesting and detailed analysis of certain features of multiple-arch dam design in such a manner as to deserve the appreciation of the members of the Profession interested in dam design and construction. In addition to the general analysis, he has given an outline of the special features pertaining to the design and construction of the two particular multiple-arch dams at Gem Lake and Agnew Lake. Essentially, however, the paper is general in its character and, correspondingly, this discussion will also be mainly of a general nature. Mr. Scheidenhelm.

The multiple-arch dam, like the flat-deck dam, belongs to what one might call the family of hollow dams. The multiple-arch type differs from the flat-deck type (of which the Ambursen dam is the most familiar) primarily in the method of transferring the water load to the supporting buttresses. Both types involve relatively high unit costs for the concrete work required. Accordingly, they find greatest favor in situations where it is proposed to build dams of concrete, but where the cost of cement is high. The dams at Gem and Agnew Lakes are cases in point, for there the average cost of cement delivered at the site of the work is reported to have been $7.50 per bbl. Apparently, only rock-fill dams, but not solid masonry dams, were considered as alternatives for multiple-arch dams for the sites mentioned. It seems

* New York City.

Mr. Scheidenhelm.

evident, however, that, had solid masonry dams been built, the high cost of cement would have more than offset the relatively high cost of the form work required for the multiple-arch dams.

As to hollow dams, there appear to be two principal points which require careful attention, namely, the protection of reinforcing steel and the tendency to skimp the sizes of the structural members. In the first respect, the multiple-arch type has a decided advantage over the flat-deck type, in that it requires a much smaller quantity of steel. Consequently, there is so much less steel which might be placed in danger of corrosion, and it is so much the easier to protect the steel. Again, the multiple-arch dam permits the use of longer spans, say, from 20 to 50 ft. between centers of buttresses, as compared with corresponding spacings for the flat-deck type of from 15 to 30 ft.

On the other hand, the cost of form work for the multiple-arch dam is somewhat greater than that for the flat-deck type, and, what is more important, the multiple-arch dam is more susceptible to difficulties and damage in case of settlement of the footings. The arches cannot withstand as much racking as flat-deck slabs. The author points out that he provides struts to take care of the unbalanced arch thrust which would result if one or more arches or bays were to fail. Of course, a dam is intended not to fail, and, hence, one is concerned more with precautions against failure than with measures to be taken in case of failure. Even from the latter point of view, however, the value of the struts would be problematical were they, in case of failure of the corresponding arches, to be subject to the impact of floating ice, tree trunks, or other débris. Moreover, it would seem that the struts might become overloaded due to blocking the passage of such débris, and thus induce failure in lateral flexure.

After all, it appears to the speaker that, in the case of the multiple-arch dam, one must insist on unyielding foundations. Fortunately, there seems to be no deficiency in bearing value on the part of the foundations of the dams on Rush Creek.

It is not surprising to find that the use of the multiple-arch dam dates far back of that of the flat-deck type, at least in so far as the construction of the latter in reinforced concrete is concerned. The most conspicuous example is that of the Meer Alum multiple-arch dam, built in India a century or more ago. This dam, referred to by Mr. Flinn, has a height of about 45 ft., and involves astounding span lengths, varying from 70 to 147 ft. The arches are plain, and are built of brick in lime mortar. New South Wales boasts of a dam, the Belubula, in which the spandrels of the arches are filled in with masonry, thus making impossible radial action of the water pressure.

In the United States there are, perhaps, a dozen multiple-arch dams, the most familiar of which heretofore have presumably been those built under the design and direction of John S. Eastwood, M. Am. Soc. C. E. The Eastwood multiple-arch dams seem to differ from those

FIG. 19.—LOS VERJELS DAM, NEAR OROVILLE, CAL.

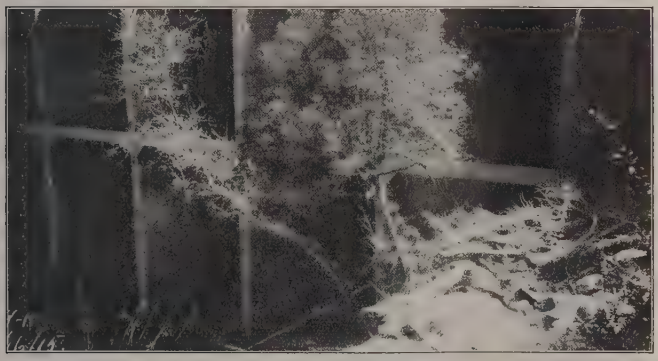


FIG. 20.—DOWN-STREAM VIEW OF LOS VERJELS DAM.

FIG. 21.—DOWN-STREAM VIEW OF NEW BIG BEAR VALLEY DAM.

designed by the author essentially as regards the treatment of the arches near the crest. Mr. Jorgensen uses an elliptical, rather than a circular, form for, say, the upper 15 ft. of each arch or panel, whereas Mr. Eastwood apparently prefers to retain circular arches, even near the crest, and, to this end, has made the arch axes vertical for the upper portions of the panels of certain of his dams. Of this design are the dams at Hume Lake and Bear Valley, California. Mr. Scheidenhelm.

The speaker has not had the privilege of visiting either of the dams described by the author, but has had the opportunity of inspecting the 60-ft. high multiple-arch dam of the Los Verjels Land and Water Company, near Oroville, Cal. This dam is illustrated in Figs. 19 and 20. The former is a view along the spillway crest. Incidentally, it emphasizes the lack of adequate spillway capacity. This is shown by the erosion which is apparent at the opposite bank.

Fig. 20, a view from down stream, shows the buttresses and bracing to be relatively slim, to such an extent, in fact, as to afford to the eye no impression of stability.

The Bear Valley Dam, Fig. 21, makes quite a contrary impression, and, though the product of the same designer, governed as he apparently was by the financial ability of the respective clients, is of pleasing appearance and proportions. Even in this case, the speaker would prefer a system of bracing between buttresses consisting of relatively deep (vertically) beams or walls. Such deeper beams would certainly afford greater stiffness, and, moreover, would tend toward a better distribution of the load on the footings. In other words, with the buttresses well braced, any deflection or settlement of the different footings would, of necessity, be practically equal, and hence the firmer, less yielding, foundation material would, as is desirable, take more load per unit of area than the less firm portions. The last-mentioned advantage of the deeper bracing beams is, to be sure, of little value in foundations where the deflection under load is inappreciable.

Turning now to the details of the paper, the speaker is inclined to think that the multiple-arch dam may properly be applied even more widely than the author claims, for he, by implication, limits its utility to locations where "the foundation is solid rock." Nevertheless, the multiple-arch dam is not entirely free from the ills of other dams. More particularly the speaker cannot agree with the general statement of the author that "there is no hydrostatic uplift to amount to anything acting on a dam of this type." Although not obsessed with such a fear of uplift pressure as is shown by some engineers, the speaker insists that the possibility, and often the certainty, of uplift pressure must be recognized.

In the absence of more definite information, the allowance made for uplift pressure must be, of necessity, a matter of judgment. Certainly, however, in the case of a foundation material which, where

Mr. Scheidenhelm.

unbroken, is impervious, but contains approximately horizontal laminations, it must be admitted that uplift pressure may exist in such laminations or seams. Under such circumstances, the effect of the uplift pressure would be practically the same as if the footings of the buttressed hollow dam were continuous, with uplift pressure existing in the plane of contact of the footings with the surface of the foundation material. Manifestly, with rock of the characteristics previously suggested, a buttressed hollow dam, such as the multiple-arch dam, has no advantage over the solid masonry dam, except to the extent that, by reason of the details of construction, it may be a simpler matter adequately to drain the underlying foundation material and thus relieve any uplift pressure.

As regards sliding, also, the speaker feels that it is unwise, and, in some instances, has been fatal, to consider a dam apart from its underlying foundation material. The foundation material, or substructure, must act as a unit with the dam body, or superstructure, if the desired object of constructing a safe dam is to be attained. The speaker does not intimate that either of the multiple-arch dams described by the author is in the least unsafe as regards either uplift pressure or sliding. However, he does feel, for instance, that it is not sufficient to aim for a safe shearing resistance at, or near, the base of a buttress, but believes that it is fully as important, and in many cases even more difficult, to design so as to obtain safe resistance to shear or to sliding within the foundation material itself at planes or laminations just below the base of the footings.

That these considerations are not far-fetched is shown by the failures of such dams as those at Austin, Tex.; Lock 26, Ohio River, and Austin, Pa. In the first two, at least, sliding took place within, and not on, the foundation rock.

The speaker has not found in the paper any specific description of the foundation material, but the author characterizes it as "bed-rock worn clean by glacial action", and this statement in itself leads one to believe that the foundation conditions are fortunate, and that probably no harmful laminations exist.

Finally, the speaker believes it would be interesting to others, as well as to himself, if the author were to share with his fellow engineers, somewhat more specifically and in more detail, his views as to the effect of the time interval (elapsing between the occurrence of maximum and minimum temperatures) as regards lessening the formation of temperature cracks.

Mr. Wegmann.

EDWARD WEGMANN,* M. AM. SOC. C. E.—Although the multiple-arch dam has only lately come into prominence, this type was adopted for the Meer Alum Dam, in India, more than a century ago. This dam,

* New York City.

Mr. Wegmann.

which is about ½ mile long, is built in plan on a curve, convex up stream, and consists of twenty-one smaller arches, having spans of from 70 to 147 ft. The dam is 39 ft. high and has vertical faces on both sides, except near its crest, where the down-stream face is stepped up stream, in order to reduce the top width.

The Belubula Dam, in New South Wales, was built about 1898. It is 431 ft. long, and has a maximum height of about 60 ft. The lower part consists of a mass of concrete from 1 to 23 ft. deep. The upper part is of brickwork, and has a height of 36 ft. 9 in. In the central part of this brickwork there are six buttresses, 28 ft. from center to center. Between these buttresses there are five vertical, elliptical arches, inclined down stream at an angle of 60 degrees. The spandrels between the arches are filled with concrete which covers the crown of the arches to a depth of 12 in., and joins the side-walls of the dam, which are of concrete. As far as the speaker knows, these two multiple-arch dams are the only ones of this type which were built until recent years.

In 1897, Henry Goldmark, M. Am. Soc. C. E., proposed to build a multiple-arch dam of concrete at Ogden, Utah.* This dam was to be 369 ft. long and 105 ft. high above the foundation at the center of the valley. According to Mr. Goldmark's plan, the dam was to consist of a number of piers, 16 ft. wide and 32 ft. apart in the clear. Alternate bids obtained for building the dam in the manner described, or, as an ordinary "gravity dam", showed a saving of from 12 to 15% in favor of the former plan. Mr. Goldmark proposed to make the thin vertical arches of the multiple-arch dam water-tight by placing a lining of sheet steel on their up-stream side.

About 1900, the speaker was retained to design a multiple-arch dam, 160 ft. high, which was to be built in Virginia, near Washington, D. C. According to the preliminary estimates, this type was found to be about 15% cheaper than a gravity section. Water-tightness was to be insured by coating the up-stream side of the dam with asphalt. Owing to the practical difficulties which might have been involved in building a multiple-arch dam 160 ft. high—a type which, at that time, had only been used for two low dams—the speaker advised his clients to construct a gravity dam. As the promoters of the project could not secure the required financial means, the dam was not built at that time.

George L. Dillman, M. Am. Soc. C. E., has demonstrated, mathematically, the saving that can be effected by building a dam of the multiple-arch type.† He proposed to give the arches between the piers a parabolic section, so as to avoid all re-entrant angles.

* *Transactions*, Am. Soc. C. E., Vol. XXXVIII, pp. 291, 302 (December, 1897).
† *Transactions*, Am. Soc. C. E., Vol. XLIX, p. 94 (December, 1902).

Mr. Wegmann.

It is only of late years that a number of multiple-arch dams have been built.

In 1908 John S. Eastwood, M. Am. Soc. C. E., of Fresno, Cal., built the Hume Lake Dam, in the Sierra Nevada Mountains, as a multiple-arch dam of concrete. The dam is 677 ft. long and about 60 ft. high, and consists of twelve vertical circular arches which are supported by thirteen buttresses. At each end of the dam, a wall, like a core-wall, is built into the side of the valley. In 1910 and 1911, Mr. Eastwood built a similar dam about 200 ft. down stream from the Bear Valley Dam, in California. It has a length of 350 ft. and a maximum height of 91.5 ft. above the foundation. In this case, bids were obtained for building a rock-fill dam having a reinforced-concrete curtain, and, also, for an arched gravity dam of concrete. The multiple-arch dam proposed by Mr. Eastwood was found to be the cheapest and was adopted.

A few more multiple-arch dams have been built. Two of these—the Gem Lake Dam, 700 ft. long with a maximum height of 112 ft. above the foundation, and the Agnew Lake Dam, 30 ft. high and 280 ft. long, were designed by the author. Mr. Jorgensen, however, has not only given an interesting account of how these dams were constructed, but, also, a very complete analysis of the stresses in multiple-arch dams. The speaker compliments him on his excellent paper which will be of value to all engineers who have to design such dams.

Mr. Douglas.

Walter J. Douglas,* M. Am. Soc. C. E.—About 7 years ago the speaker's firm designed and built, in the Adirondacks, the Garoga Dam, a multiple-arch structure having a 200-ft. spillway of the gravity type. The concrete arches are 60 ft. high, and their thickness at the top is 2 ft., increasing to 4 ft. at the bottom. There was no difficulty in taking care of the thrust, the foundation being of good slate, strong enough to take the concentrated load transmitted by the buttresses. The economy of the multiple arch was found to be small when its cost was compared with that of the gravity section used for the spillway. The speaker thinks that in most cases where the cost of skilled labor is high and material is cheap, the ordinary gravity type is about as cheap as the multiple arch. The difficulties of constructing thin arches, such as those shown in the paper, often more than offset the saving in material.

The speaker's firm has made a great many designs of dams since the Garoga Dam was built, but has only constructed one other of the multiple-arch type. This was the Peck's Lake Dam, built near the same locality as the Garoga Dam. The height of the arches, above the surface of the ground, varies from 25 to 45 ft. As far as the

* New York City.

speaker knows, this is the only multiple-arch dam which has not been built on a rock foundation. The foundations were of slightly indurated sand and gravel, but not hardpan. The pressure is distributed by reinforced concrete slabs at the toe of the buttresses. As a security against horizontal movements, the arches penetrate about 10 ft. into the ground, where they are loaded uniformly by the reaction of the soil, and, therefore, they are catenary in tension. This part of the arches is heavily reinforced with steel rods, which are well bonded into the buttresses. This arrangement worked out very satisfactorily. Mr. Douglas.

The dams described by the author are very interesting, but, the speaker believes, they are not conservatively designed. All engineers who are interested in the development of reinforced concrete, as applied to structures other than buildings and bridges, know that every once in a while something "gets by." Tanks have been built in recent years, which are 4 in. thick at the top, 8 in. at the bottom, and more than 100 ft. high, but the speaker thinks that is no justification for building similar ones. He believes that both the arches and the buttresses of the dams shown in the paper are unnecessarily thin.

The speaker cannot see how the triangular girder, mentioned on page 866, will take up the unbalanced thrust of the arches if one of them should fail, and would ask for more detailed information on this point.

The most serious objection which can be raised against multiple-arch dams is the fact that failure of one unit will destroy the stability of the whole structure. The speaker believes that an occasional abutment buttress, similar in purpose to the abutment piers of arch bridges, consisting of a series of spans should be introduced in the design of multiple-arch dams having a large number of arches.

On page 865, the author says:

"The total vertical load on the section shown in Fig. 6 is seen to be 6 332 tons, and the horizontal water pressure, 5 062 tons, both on a 40-ft. span. If the coefficient of friction is taken at 0.75, it is seen that the actual shear along the base amounts to only $5\,062 - 6\,332 \times 0.75 = 313$ tons. There is considerable steel in the section to help take up this shear, and, therefore, it was not deemed necessary to eliminate the shear entirely."

This statement, the speaker believes, is incorrect. The shearing resistance of an elastic solid is an elastic attribute of the material. The deformation caused by shearing forces, if within proper limits, will disappear after the actuating forces have been discontinued. The frictional resistance is of an entirely different nature, and can act only after the section has failed under shear. The resisting friction can be developed only by minute, but measurable, movements of adjacent planes. This movement is progressive, that is, the dislocation caused by the forces resisted by friction will not disappear when the

Mr. Douglas.

structure is relieved of load, and a second application will necessitate a further movement. In designing one should rely on shear or on friction, and not on both.

As regards the determination of the direct stresses, the speaker would state that the stress computed by the author on page 866, acting on planes at right angles to the resultant, is not necessarily the maximum compressive stress, and that methods are now available for computing the principal stresses in a buttress quite accurately.*

The speaker has made the foregoing criticism because, as the author correctly states on page 851, "the stresses and dimensions can be calculated with great accuracy", in multiple-arch dams, although such calculations are almost impossible in earth or rock-fill dams. The determination of the stresses is subject to many uncertainties, even in gravity dams. The design of this latter type, however, is being developed along scientific lines with very satisfactory progress, and the gravity dam has the advantage of a great number of precedents over the multiple-arch dam.

In the case of solid impervious rock foundation, the speaker decidedly prefers the gravity dam, and doubts whether material saving, if any, can be obtained by the use of the multiple arch, even if this latter would have a considerably smaller volume. The unit price of masonry for the multiple-arch dam is out of all proportion to that of the gravity dam, as illustrated by the price of $22 per cu. yd., given by the author.

If the foundation is fissured rock or other good but pervious material, the multiple-arch dam, generally, should be the more economical type, because the volume of the gravity dam must be increased, in order to meet the uplifting action of the water.

Mr. Duryea.

EDWIN DURYEA,† M. AM. SOC. C. E.—The speaker's home and most of his professional practice are in California, and he is familiar with the general climatic and hydrographic conditions throughout the State and in the Sierra Nevada Mountains, though not with those of Mono County, in which are built the two dams forming the subject of this paper. However, hydrographic conditions are known to be quite uniform throughout the high Sierras of California, and presumably in Mono County they do not differ materially from those of the parts with which the speaker is familiar.

A previous speaker has expressed doubts as to the adequacy of the spillway arrangements of these two dams to pass safely such intense floods as might be expected to occur at long intervals from such a small drainage area as 22½ sq. miles. In most parts of the United States, such apprehensions would be justified, but not in the high Sierras of California.

* See the papers of Levy, Unwin, Hill, Ottley, and Brightmore.

† San Francisco, Cal.

Mr. Duryea.

Throughout that State practically all the year's precipitation occurs during the winter, from about October to about March, inclusive, with hardly any (and none such as will cause floods) during the warmer months of April to September. Hence, in the high Sierras, the year's precipitation is practically all in the form of snow, which accumulates throughout the winter to depths of even 20 ft., or more, settles and packs until melted gradually by the increasing temperatures of the spring and summer, and passes off as stream flow in the form of slowly rising floods, which usually begin in April, culminate in June, and terminate about the middle of July. It is this condition of winter rains in the valleys, which soak the ground and start the growth of the crops, in conjunction with the snow stored in the high Sierras, which by its gradual melting furnishes water for irrigation throughout the growing season of April to September, that causes the very great success of irrigated agriculture in the Great Valley of California.

On the drainage area above these two dams (at elevations of from 9 000 to 12 000 ft. above sea level), and even at much lower elevations, this condition of stream flow from the slow melting of the snow must be marked and invariable; and on that area there can be no direct connection between excessive storm intensities and excessive flood intensities.

The speaker has examined many dams in the high Sierras of California and, at first, was strongly impressed by the apparently very inadequate provisions for spillway capacity and "free-board" of those dams, as judged by necessities elsewhere. Soon, however, he realized that provisions which would have been very inadequate and unsafe in most other regions are quite safe in the high Sierras.

A previous speaker has mentioned the small number of arch dams as yet built in the world, placing the number at only about 15. The speaker has built an arch dam which may not be included in that number. It is the Goodwin Dam, on the Stanislaus River, not far from Stockton, Cal., and serves to raise and divert the waters of that river for the irrigation of the South San Joaquin and Oakdale Irrigation Districts, which together comprise about 144 000 acres. The dam consists of two arches with a buttress between them; there is also a small buttress at the south end of the dam to replace a rock abutment which was badly shattered by the blasting of a canal. The south arch is 299.4 ft. in crest length, with a maximum height of 78 ft.; the north arch is 159.8 ft. in crest length, with a maximum height of 42 ft. The buttress between them is on a ridge of rock (slate) near the middle of the river, and has a maximum height of masonry of only 22 ft.

The two arches have radii (of their vertical up-stream sides) of 135 ft., and are proportioned for "pure arch action" by the ordinary arch formula and for maximum arch stresses of 300 lb. per sq. in. (43 200 lb., or 21.6 tons per sq. ft.). Except that the minimum arch

Mr. Duryea.

thickness (in the upper parts of the arches) was fixed arbitrarily at 8 ft., the arches were thickened 4 ft. arbitrarily at the rock surface (this thickening decreasing to zero 8 ft. above), as a safeguard against possible erosion by the very turbulent waters of the pool just below the dam. The crest was made of a somewhat unusual shape to permit larger flows over it without reducing its strength, and the arches were reinforced against temperature stresses by steel bars, near their crests and for 6 ft. below, near their up-stream and down-stream sides. The thickness of the south arch, 59.8 ft. below its crest (just above the thickened base), is 12.2 ft.; that of the north arch, at the corresponding point, 24.5 ft. below its crest, is 8.0 ft.

The arches are proportioned (aside from the exceptions noted) for 300 lb. per sq. in. and for the maximum arch thrusts caused by floods up to 25 ft. in depth over the crest, making allowance, however, for the relief of the arch pressure by the variable "back-water" against the down-stream sides of the arches. Two measured floods are known (in 1907 and 1911) which would have caused depths on the crest of the dam (had it then been built) of about 15 ft. The dam was completed in 1912, and since then several floods of about 6 ft. in depth have passed over it. The drainage area above the Goodwin Dam comprises about 928 sq. miles of mixed low and high Sierras, from an elevation of 350 ft. above sea level (the crest of the dam) to an extreme height of more than 10 000 ft.

The first requirement in any dam should be certain safety, but the next should be economy; and the usual reason for designing an "arch" dam is that thus considerable saving below the cost of a "gravity" dam often may be effected, without (at least in the judgment of the designer) any sacrifice in safety. The Goodwin Dam is a good example of such a saving in cost. Six other dam sites were surveyed, three up stream and three down stream from the Goodwin site. Estimates of cost were made for "gravity" dams on all seven sites, and for three other plans of arch dams on the Goodwin site. By the choice of the Goodwin site and the two-span arch dam adopted and built, the saving effected was more than half that of the most obvious procedure—the raising of an existing gravity dam 1¾ miles up stream, already the property of the two Irrigation Districts, the enlarging of the intervening 1¾ miles of their existing canal on the north side of the river, and the building of the same length of new canal on the south side.

In 1903, the speaker designed two other arch dams for California streams, neither of which, however, has been built.* The larger one was to have been 138 ft. in maximum height, and made up of eight semicircular arches, supported on intervening buttresses 50 ft. from center to center. The backs or up-stream sides of the arches were to

* *Transactions*, Am. Soc. C. E., Vol. LIII (December, 1904), pp. 172-175.

have been vertical, with radii of 26.5 ft. The smaller dam was to have been 30 ft. in maximum height, with eight spans of 19½ ft. from center to center of buttresses. The "deck" or up-stream side of the dam was to have been a plane with an inclination of 0.64 vertical to 1.00 horizontal, in order to distribute the foundation pressures more evenly over a rather weak rock foundation. The crown thickness of the deck was to have been 2 ft., and the radii of the inclined semicircular arches forming the under side of the deck were to have been 8¼ ft. Mr. Duryea.

Mention is made in the paper, and by a previous speaker, of the freedom of multiple-arch dams (or other thin dams) from "uplift" forces. Such forces are quite certain to exist in greater or less degree under all thick masonry dams. The speaker recently had an opportunity to measure the uplift forces existing under parts of the base of a high masonry dam (the La Boquilla Dam, in the State of Chihuahua, Mexico, of 244 ft. maximum height). The measurements extended over a period of 22 months, and for depths of water behind the dam (above the foundation level) of from 140.3 to 213.6 ft.; and comprised measurements in thirty-nine holes, aggregating sixty-three or more hole measurements of the uplift. He hopes soon to present these measurements in full to the Society; and will only say now that, although the foundation of that dam, in its natural condition, was a very good limestone, with but few and minor defects (which were all repaired with great care and cost by cement pressure grouting), still the measurements of the pressures show the existence of "uplifts" which, in general, lie on straight lines decreasing from (at the "heel" of the dam) one-third of the depth of water in the reservoir above the foundation plane, to zero at the "toe" of the dam. Incidentally, the dam had been designed to withstand uplifts varying from two-thirds of the depth of the water at the heel to zero at the toe.

Regarding the use and safety of arch dams, one general thought is worthy of serious consideration: Several (only a small proportion, however) "gravity" masonry dams have failed. It is true that these failures were probably always from faults perceived afterward to be due to recognized poor design or construction, and that they would not have occurred had the design and the construction of the profiles and the foundations been in accordance with recognized good practice.

On the other hand (even though most arch dams are much thinner than gravity dams, and though there should be no reason to suppose their profiles and foundations to have been in general designed and constructed more carefully), the speaker does not recall, among the few arch dams as yet constructed, even a single failure.

It is realized that such reasoning is not conclusive, but, nevertheless, it appears to the speaker at least probable that the "arch" dam possesses greater reserves of strength and safety than the "gravity" dam.

Mr. Nishkian.

L. H. NISHKIAN,* ASSOC. M. AM. SOC. C. E. (by letter).—The author has rendered to the Engineering Profession a much needed service by opening up for discussion the subject of multiple-arch dams.

There is no question that, in the future, dams of this type will prevail over other types in many locations, and, therefore, a detailed discussion of methods of design and construction is timely and profitable.

At first glance the structural design of a multiple-arch dam seems to be quite simple, requiring the design of a concrete arch under definite forces, of a buttress to uphold the arch, and of a foundation to hold the buttress in place.

On closer study, many problems arise. The upper and lower ends of the arch barrel are difficult of even roughly approximate analysis, and the designer has to depend largely on his judgment. The weight of the arch between the buttresses is carried partly on the latter and partly on the lower end of the arch. In designing the arch, the author has considered its weight component normal to the buttresses as being carried by arch action to the buttresses, and, presumably, the component parallel to the buttresses as being carried by direct column action down to the footings at the lower end of the arch barrel. In the design of the buttress and foundation, however, it has been assumed that the entire weight of the arch is acting, through its center of gravity, on the buttress.

The writer agrees with Mr. Jorgensen in regard to the method of designing the arch ring, but cannot agree with him in throwing the entire weight of the arch on the buttresses. It appears to the writer that only the weight component normal to the buttress should be considered as acting on it. These two assumptions are widely at variance, and result in a large difference, both in the point of application and in the line of action of the resultant force on the buttress.

The author finds an upper and a lower limit for the direct compression stresses in the buttress. It will be found, however, that the stress produced by the normal component of the arch thrust on the buttress is equal to, or greater than, the maximum stress given in the paper. In this case, the arch thrust, due to water pressure at a depth of 80 ft., is approximately $80 \times 62.5 \times 23.08 = 115\,000$ lb., and the thrust, due to dead weight of arch ring, is approximately $150 \times 3.7 \times \cos. 50° \times 23.1 = 8\,000$ lb., or a total arch thrust of 123 000 lb. This will result in a force of $123\,000 \times 2 \times \sin. \frac{119° 57'}{2} = 213\,000$ lb. per lin. ft. on the buttress. The buttress is here 4.25 ft. thick; therefore, we have a stress of $\frac{213\,000}{4.25 \times 12 \times 12} = 348$

* San Francisco, Cal.

lb. per sq. in. Although in no sense dangerous, this is in excess of the stress of 317 lb. per sq. in., given by the author as the maximum existing in the buttress. One must be careful in using average values of stresses where the average is for a large area. Mr. Nishkian.

The author has made the section of the arch normal to the up-stream edge of the buttress, circular. As is pointed out in the paper, the pressures are not uniform on the arch ring, with the result that, near the top, it is necessary to move the arch out 5 in. at certain points, in order to make its center line coincide with the line of pressure. To avoid this difficulty and maintain a constant shape from top to bottom, it is only necessary to make a horizontal section of the arch circular in shape. A normal section will now be an ellipse, and it will be found that the center line of the arch ring and the line of pressure will coincide very closely for all depths.

It will be necessary to use a three or five-centered curve for the form ribs, and this will entail an extra expense, as the circular form ribs will have to be made in sections also. Furthermore, with a horizontal section circular, the up-stream and down-stream edges will be concentric, with a uniform radial thickness of arch ring. This will result in the normal section of the arch ring being much thicker at the buttresses than at the center, giving it greater stability.

The author has provided a light but very effective method of stiffening the buttress walls, and it is worthy of more general use.

Gardner S. Williams,* M. Am. Soc. C. E. (by letter).—No discussion of the multiple-arch dam would be complete without mention of the famous Meer Alum Dam,† near Hyderabad, India, built about 1800, which is made up of a series of full-centered vertical brick arches with radii varying from 70 to 147 ft. These arches are arranged in the arc of a circle, having a crest length of about ½ mile and a height of about 40 ft. This structure appears to be the dean of multiple-arch dams. Mr. Williams.

On September 1st, 1902, the writer tabulated the bids received by the Ithaca Water Works Company, and a few days later awarded the contract for the construction of the so-called Six-Mile Creek Dam, near Ithaca, N. Y. This dam embodied in its design a section of the inclined multiple-arch type, and is believed to be the first structure of the kind for which a contract was ever let.‡

Prior to the design of this dam, which was made by the writer in the early part of 1902, plans and specifications for an inclined multiple-

* Ann Arbor, Mich.

† *Transactions*, International Engineering Congress, 1915, Waterways and Irrigation, p. 719.

‡ The Six-Mile Creek Dam was discussed in *Transactions*, Am. Soc. C. E., Vol. LIII, and the multiple-arch section is particularly described on page 195 thereof.

Mr. Williams.

arch dam were prepared by Henry Goldmark, M. Am. Soc. C. E., for the Pioneer Power Company, of Ogden, Utah,* the suggestion of this design being due, the writer is informed, to George H. Pegram, President, Am. Soc. C. E. The inclination of the arches was about 25° from the vertical. Subsequently to the work of Mr. Goldmark and prior to 1902, the multiple-arch dam was investigated by James H. Fuertes, M. Am. Soc. C. E., and by the late Emil Kuichling, M. Am. Soc. C. E., independently, but neither of their designs came to the constructive stage.

Since the construction of the Six-Mile Creek Dam, dams of the multiple-arch type have been designed and built under the supervision of the writer in Michigan, Minnesota, and Iowa.

The principles of design laid down by the author differ somewhat from those followed by the writer, and appear to transgress the rules of economical design, since the structure, as described by the author, does not follow the equilibrium curve. As is well known, the curve of equilibrium for an arch under uniform normal pressure is a circle, and, therefore, since the inclined arch may be conceived as made up of an infinite number of infinitesimally thick horizontal arches, the horizontal section of the arch should be a circle and not an ellipse, as seems to have been contemplated by the author. A section of the arch normal to the axis will then be elliptical, and the inclination should be chosen so that this ellipse approaches most closely to the hydrostatic arch for the condition of most constant load. When designed in accordance with these principles, the arch is subjected to the minimum of bending stress, and requires the least material. An increasing thickness of the arch section to the bottom of the dam is hardly necessary, as over the lower one-third, in ordinary cases, the beam action is such as to relieve to a considerable degree the arch action, so that the shell may very properly be made of uniform or even diminishing thickness from the point of one-third the height down.

None of the dams built by the writer has been on a rock foundation, and although it is necessary to be assured of the carrying capacity of the material under the buttresses, this does not preclude the use of such structures on sand, gravel, or clay.

Mr. Howson.

George W. Howson,† Assoc. M. Am. Soc. C. E. (by letter).—Mr. Jorgensen has presented a very excellent paper on the design and construction of multiple-arch dams. The method of working out the design shows much careful study, and appears to be the last word in the design of dams of this type.

There are several phases of the paper, however, which hardly seem to be in accord with the writer's experience, and there is a general

* *Transactions*, Am. Soc. C. E., Vol. XXXVIII, pp. 290 to 297.

† San Francisco, Cal.

tendency to compare the multiple-arch with the rock-fill dam. These are two entirely different types, each having its own advantages. Mr. Howson

In general, it may be stated that a rock-fill dam may be built on any foundation suitable for a multiple-arch structure; but the reverse is not true. As an example, a rock-fill dam is best adapted to a site where the underlying bed-rock across the floor of the canyon is covered by, say, from 20 to 30 ft. of over-burden. Here it may be necessary to carry only the thin cut-off wall down through the overlying material into solid rock; and the great bulk of the dam, the loose drop fill, may be built on the over-burden. It must be known, of course, that the foundation under the rock fill is suitable, though it may not be a foundation on which the buttresses of a multiple-arch dam should be built. Mr. Flinn has brought out this point in comparison with a solid masonry dam, but his comparison of economy relative to the foundation is even more applicable in the case of a rock-fill structure.

It is then seen that the problem of choice between rock-fill and multiple-arch types presents itself only at a site where a good foundation for the buttresses is exposed, or is found within a limited depth of excavation. Here the final economic balance may be in favor of either type, depending on the particular cross-section of the canyon, the transportation problem, and the general topography of the site. The writer, however, believes that, in a majority of cases where it is possible to consider a choice of types, the relative cost of a multiple-arch dam and a rock-fill dam will be in favor of the former.

Mr. Jorgensen is correct when he states that the water-tightness of a dam depends to a much larger extent on the quality of the concrete in the arches than on the thickness of the latter. The Strawberry Rock Fill Dam, built recently by the Sierra and San Francisco Power Company, may be said to be bottle-tight, although it depends on a reinforced concrete apron only 18 in. thick, under a 130-ft. head, which thickness tapers to 9 in. at the crest. A multiple-arch dam should be built only under the most competent engineering supervision. A few failures of dams of this class due to poor construction would condemn all such structures, in the public opinion.

L. R. Jorgensen,* M. Am. Soc. C. E. (by letter).—Throughout the discussion there has been a tendency to compare the cost of multiple-arch dams with that of gravity dams. As a general rule, it can be said that the multiple-arch dam is the cheaper, the more expensive the building material. For dams of moderate height, it is probably always cheaper, but not for heights of more than 130 ft. The only sure way of finding out is to estimate the cost of each type for any given location. Mr. Jorgensen.

The system of struts has not been designed to help to prevent settlement or sliding of the buttresses; it cannot do that, and it is not

*San Francisco, Cal.

Mr. Jorgensen.

necessary, for, in these particular cases, the rock foundation is possibly twice as strong in shear and crushing as the concrete in the buttresses. The struts are to stiffen the buttresses, and to transmit any unequal arch thrust which normally might exist at any place, due, for instance, to the adjacent arches being built, during periods of different temperatures, into the side-hill where the struts are anchored. The two struts close to the arch (Fig. 4) will also transmit to the side-hill the abnormal unbalanced thrust which might occur should one arch accidentally fail, thereby preventing a complete collapse of the dam. Should the struts fail at the same time as the corresponding arch—which is probable—the unbalanced thrust would have to be taken up in tension by the steel in the line of struts on each side of the break. The eight 1-in. square rods should be able to hold 400 000 lb., as there are no laps between the knee-braces belonging to the different buttresses. The triangular girder mentioned on page 867, simply stiffens the up-stream ends of the buttresses so that they will stand up between points of application of the struts. The struts then transmit the unbalanced thrust to the side-hill, either by tension or compression, or by both.

Mr. Blackwell's criticism as to spillway capacity has been answered by Mr. Duryea. There occurs no precipitation of such quantity as to cause flood when the reservoir is full. During the season of maximum ice pressure—that is, in late winter or early spring—Gem Lake Reservoir will be nearly empty. Agnew Lake Reservoir is to be used as a forebay, and the daily fluctuations will break up the ice; therefore, it has not been deemed necessary to design these dams for ice pressure.

In comparing the three forms of multiple-arch dams mentioned by Mr. Flinn, the writer regards the one with the sloping up-stream face as the best and only one to consider. In this construction, the water pressure acting on the sloping face is the largest single factor contributing to stability, and this factor would be absent in a dam having a vertical face.

As to the leakage of different types of dams, this will depend much more on the care with which they are constructed than on their type and design.

The writer agrees with Mr. Scheidenhelm, that, after all, in the case of multiple-arch dams, one must, for perfect safety, insist on an unyielding foundation.

As an example of the influence of the time factor, let it be assumed that an arch is subjected suddenly, or during an interval of a few hours, to a temperature drop, penetrating through the arch material, of, say, 80°, sufficient to cause a maximum tension in the concrete of, say, 300 lb. per sq. in. If this same temperature drop of 80° was brought about during an interval of 20 or more days, the

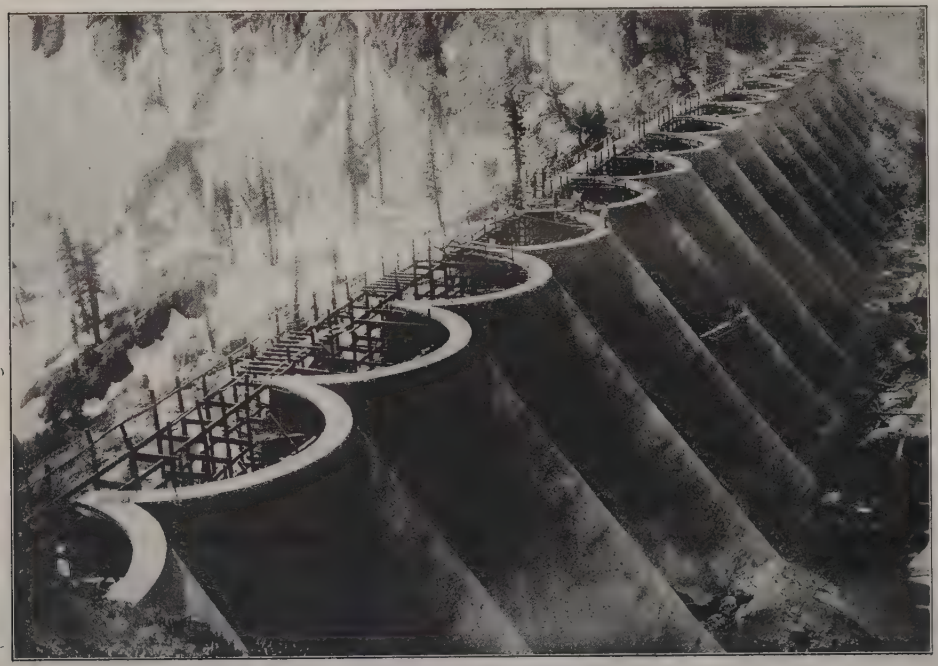


FIG. 22.—GEM LAKE DAM.

Mr. Jorgensen.

tension in the concrete would only amount to about 100 lb. per sq. in., and, naturally, the tendency to form cracks would be much less than in the first case. Daily temperature changes will hardly penetrate the arch sufficiently to have any practical effect, but seasonal temperature changes will, and here is where the action of the time factor tends to lessen the tendency to form cracks by making the concrete stretch more for the same stress, because the change has been slow. The publication by Mr. F. R. McMillan, referred to on page 859, gives detailed results of numerous tests of this nature, and is very interesting.

The multiple-arch dam at Ogden, Utah, referred to by Mr. Wegmann and Mr. Williams, is of unusually conservative design, and it is no wonder that the bid price was only from 12 to 15% lower than the bid price on an ordinary gravity dam for the same site, as Mr. Goldmark's design surely has a very much larger factor of safety than the ordinary gravity dam.

Mr. Douglas says, "The unit price of masonry for the multiple-arch dam is out of all proportion to that of the gravity dam, as illustrated by the price of $22 per cu. yd. given by the author." This high price, however, is due to the high cost of the material at this particular place (Gem and Agnew Lakes), and not to the type. A multiple-arch dam in an accessible location, and where conditions are ordinarily favorable, can be constructed for $13 per cu. yd., and this price may possibly include the steel reinforcement.

Mr. Douglas is correct in stating that frictional resistance can only act after the section has failed under shear. One should rely on shear or friction. The maximum unit shear of approximately 106 lb. per sq. in. existing at Elevation 80 below the crest can hardly be considered excessive. In this case the area of the arch ring has been added to the area of the buttress proper, as both must act to resist shear at the same time. The fact that there is a very large vertical load should be of some comfort, as it improves the ability of the buttresses to withstand actual shear.

The stress on the buttresses is given between two limits on pages 865 and 866. The following formula (which can be found in some handbooks) gives the maximum stress on any horizontal plane more definitely:

$$\text{Ideal maximum stress} = \frac{m-1}{2m} \times \text{unit vertical compression}$$

$$+ \frac{m+1}{2m} \sqrt{(\text{unit vertical compression})^2 + 4\,(\text{unit horizontal shear})^2}$$

(m = Poisson's ratio = 5 for concrete). Then, at Elevation 80,

$$\text{Ideal maximum stress} = 0.4 \times 192 + 0.6 \sqrt{192^2 + 4 \times 106^2}$$
$$= 76.8 + 171.6 = 248.4 \text{ lb. per sq. in.}$$

Mr. Jorgensen.

It is true, as pointed out by Mr. Nishkian, that the stress produced by the normal component of the arch thrust on the buttress is greater than the stress (317 lb.) produced by the resultant. This, however, is the case only toward lower elevations, where the "column" is very short; the normal component of the arch thrust diminishes rapidly toward higher elevations. The ideal maximum stress of 248.4 lb. per sq. in. diminishes less toward the upper system of struts; and, although the column ratio is about 20, there might be, and very likely is, some column action due to the ideal maximum stress.

Both Mr. Nishkian and Mr. Williams point to the elliptical arch as being the most correct form. Thus far, the writer has contended that the additional work of making a three-centered arch would outweigh any possible gains; however, he will soon see how the elliptical arch works out on a multiple-arch dam structure with the construction of which he is connected.

The writer cannot agree with Mr. Williams that the arch can be of uniform, or diminishing, thickness downward from the point of one-third the height of dam. Even though the stresses may not be excessive, the penetrating power of water increases with the depth, and, therefore, the thickness of the wall should increase logically. It must be kept in mind that even a sweat on the down-stream face will produce icicles in cold weather, and it is not yet known to what extent, if any, this will damage good concrete. It is known that it damages poor concrete. Good concrete is fairly water-tight, and gunnite is remarkably so, but, nevertheless, it requires great care to construct a water-tight structure. Some of the arches on the Gem Lake Dam are absolutely tight, some of them sweat in places, and a few drip in places. A few small springs have formed behind the dam, and a trickle of water comes under one arch; but, all told, the total leakage is very small.

Some tests were carried out in the Laboratory of the University of California on the water-tightness of plaster "shot" on the dam face with a cement gun. Several plaster slabs, from $\frac{5}{8}$ to $1\frac{1}{2}$ in. thick, made at Gem Lake, were tested with water pressures ranging from 700 to 1 600 ft., for several hours, with no moisture coming through the slab. One 1-in. slab held a head of 1 610 ft. for $2\frac{1}{2}$ hours without showing moisture, then the water pressure was raised gradually to 3 400 ft., and the specimen broke in bending, having leaked a little just before breaking.

Mr. Howson states the case correctly when he says that a rock-fill dam is best adapted to a site where the underlying bed-rock across the floor of the canyon is covered by, say, from 20 to 30 ft. of over-burden, as it is only necessary to carry a thin cut-off wall down through the overlying material.

The author thanks those participating in the discussion for the interest shown.

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Paper No. 1397

THE RECONSTRUCTION OF THE STONY RIVER DAM*

By F. W. Scheidenhelm, M. Am. Soc. C. E.

With Discussion by Messrs. J. W. Ledoux, J. K. Finch, P. P. Rutenberg, Fred F. Moore, W. S. Downs, H. L. Coburn, H. F. Dunham, Orrin L. Brodie, William Cain, Charles E. Gregory, Kenneth C. Grant, L. R. Jorgensen, Edward Wegmann, Irving P. Church, M. M. O'Shaughnessy, Joel D. Justin, Ross M. Riegel, and F. W. Scheidenhelm.

Synopsis.

This paper describes the reconstruction and strengthening of the Stony River Dam, which failed on January 15th, 1914. After the failure, on investigation of the foundation conditions and the parts of the structure which remained intact, it was found that those portions required strengthening and revision in a number of important features. The lessons indicated by the results of the investigation are significant. The dam, even as reconstructed and strengthened, is founded largely on clayey soil, thus presenting most difficult foundation conditions. It is about 50 ft. high above stream level and 85 ft. between the lowest part of the cut-off wall and the top of the parapet.

A brief résumé of the previous history of the dam, including the causes of its failure, is first given, together with a statement of the controlling geological and foundation conditions. Then follows an exposition of the treatment of the more important problems involved in the reconstruction.

* Presented at the meeting of March 21st, 1917.

The spillway capacity was increased approximately to 1 840 cu. ft. per sec. per sq. mile. The reasons for providing such unusually large capacity are given, and also a method of determining the absorption effect of a reservoir in smoothing off the peak of a flood.

The most important problem to be solved was that of giving the original structure a sufficient margin of safety against sliding. The results of tests of frictional resistance and shearing value of various soils, principally clays, are stated, and also the various schemes considered for increasing the resistance to sliding. The method adopted is believed to be new, and consists in the use of anchoring walls extending to a considerable depth into the underlying foundation soil, and, in effect, utilizing the weight of that underlying soil as well as the resistance (to horizontal movement) of the soil immediately down stream from the structure. The details of construction are unique. The same principles were applied, with important differences in detail, at the new spillway which replaced that portion of the structure which had failed or was wrecked.

Brief reference is made to the resistance of the structure—both in its original state and as strengthened—to failure by overturning.

The bearing value of clayey soil is discussed, and the results of tests made at the site are given. The anchoring wall added at the toe of the original structure was utilized to decrease the pressures of the foundation soil. The method of strengthening the original footings is described. Pressure grouting was used to remedy other faulty footing conditions.

The means of cutting off percolation and leakage through the foundation soil are described, also the provision made for drainage, both of clayey soil and of shale rock, together with the controlling considerations in these and similar cases.

The higher bulkhead portions of the original structure were housed in by curtain-walls and roofs in order to prevent serious freezing in the drainage system and to fulfill certain other functions as described.

Miscellaneous problems in the reconstruction and strengthening involved the type of structure for the new spillway, the strength of the original decks and buttresses, the underpinning of the portions of the original structure immediately adjoining the new spillway, the construction of a protecting and anchoring toe-wall at the old spillway, the

special method of constructing an anchoring wall at the heel at the outlet-gate sections, and the provision of an outlet channel.

The reliable storage capacity of the reservoir was increased by 25 per cent. This increase was made permissible by the use of special steel bars or pins acting as flash-board supports. These pins were developed for the purpose, and automatically and reliably allow the flash-boards to be swept off the spillway crests when the head-water reaches a pre-determined level.

The materials, methods, and cost of reconstruction are discussed briefly. Approximately, as much concrete was required for the reconstruction as had been placed in the original structure.

The Stony River Dam and Reservoir have again been in service since May, 1915, and the observed results of the reconstruction and strengthening are stated in connection with the corresponding features of design.

Reconstruction of Stony River Dam.

The failure and reconstruction of the Stony River Dam are of interest, not because of the height or dimensions of the structure, but because it is largely through failures and mistakes that engineers must learn the limitations of previous methods of design and construction and gather hints as to new methods to be used. Furthermore, in the case of this structure, the extremely difficult foundation conditions made the problem of providing a safe dam at that site one requiring more than usual thought and care. Finally, much interest attaches to the methods used to make safe the existing structure, that is, that portion which remained intact after the failure—it being noted that a comparatively small section of the dam actually failed.

It seems proper at this point to emphasize the fact that, until a structure has actually failed, its "safety" is relative. Seldom do any two engineers have exactly the same conception or measure as to the factor or margin of safety possessed by a given structure. Especially is this true in the case of dams.

As a preface to the description of the reconstruction and the problems involved therein, the following brief review of the previous history of the dam is pertinent. The details have been gathered from various sources, and are based on the best information available to the writer.

Previous History.

In 1911 the West Virginia Pulp and Paper Company, owner of the dam, determined to investigate the feasibility of constructing on the head-waters of the North Branch of the Potomac River a storage reservoir by which to increase the flow of the river at its Piedmont pulp and paper mill at Luke, Md., near Piedmont, W. Va. Its requirements of water for manufacturing purposes were such that during low-water seasons the normal flow of the stream was at times inadequate.

After an investigation of the available sites on the drainage area above the Piedmont mill, the owner chose the water-shed of Stony River as offering the best conditions for the construction of a storage reservoir. Stony River—which at the site of the dam drains only 11.4 sq. miles, and is therefore only a creek—lies entirely within Grant County, West Virginia, flowing into the North Branch of the Potomac River from the south a short distance above Schell, W. Va. The mouth of Stony River is approximately 26 miles up stream from the Piedmont mill. It is only near its upper end that the profile is flat enough to allow the construction of a reservoir at a practicable cost per unit of water stored.

Two sites examined in the upper reaches of Stony River were abandoned, because of interference with a seam of commercial coal in one case and the presence of quicksand in the over-burden in the other. The third and final site was chosen with the approval of Edward Wegmann, M. Am. Soc. C. E., who had been engaged by the owner as Consulting Engineer for the purpose.* This site is approximately 3 350 ft. above sea level.

An examination of the foundation conditions, deemed at the time to be adequate, was made by test pits and auger borings. After this examination it was concluded that the depth to bed-rock was so great as to prohibit the construction of a solid masonry dam, and that the surface soil in the neighborhood of the dam was not of such a nature as to warrant the construction of an earth fill dam. It had been the intention of the Company to build a dam of one of these types, but, under the circumstances, it was decided to construct a hollow reinforced concrete structure with a height of about 50 ft. above the original water surface and a length of about 1 075 ft. The resulting capacity

* *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 1069.

was approximately 1 533 000 000 gal. below the elevation of the spillway crest.

Certain companies were invited to submit plans and bids for the construction of such a dam. On the basis of the bids received,* a contract was entered into with Mr. Frederick G. Webber, President of the Webber Construction Company, for the construction of a dam according to plans submitted by the Ambursen Hydraulic Construction Company, licensor.

Work was begun by the contractor about June 1st, 1912, and carried on until February, 1913. At that time the Company took the work from the original contractor and engaged the Ambursen Hydraulic Construction Company to complete it. The latter company took over the work in March, 1913, and completed it in July, 1913.

The details of the original construction have been fully described by G. H. Bayles, M. Am. Soc. C. E., Resident Engineer for the owner during the construction.† Variations of actual conditions from those assumed in the original design and construction will be discussed later in this paper. Fig. 1 shows the dam as originally built.

The storage of water in the reservoir was begun in May, 1913, that is, prior to the completion of the dam. The reservoir, however, was not completely filled until the late autumn, and the water level had been at the spillway crest for about 65 days prior to the failure of the dam. There had been but little flow over the spillway.

The failure occurred on January 15th, 1914, the cause, in the opinion of the writer, being the washing out of the foundation soil by leakage from the reservoir under an up-stream cut-off wall of inadequate depth. It is probable that there were conclusive signs of impending trouble for several days earlier (perhaps as early as January 10th), in the way of water flowing through weep-holes in the floor or footing of the dam at or near the section where failure finally occurred. With proper attention on the part of the operating attendant at the dam, the reservoir could probably have been drained in time to avoid the failure which occurred.

It is not the purpose in this paper to discuss the failure in detail, inasmuch as it has been described adequately in articles in engineering periodicals.‡ For the sake, however, of making clear the description

* *Engineering News*, September 5th, 1912.

† *Engineering News*, January 22d, 1914.

‡ *Engineering News*, January 22d, 1914; *Engineering Record*, January 24th, 1914.

of the design, and the work involved in the reconstruction, Figs. 2, 3, and 4 are submitted, showing the portion of the dam which failed.

As a matter of personal interest, the writer visited the site as soon as practicable after the failure, and spent portions of January 18th and 19th, 1914, there. At the time of his inspection the main outlet-gate in the dam had been opened, and the reservoir had already been completely drained, so that Stony River was flowing in its normal channel at the center of the valley, and not through the break, which occurred at a higher level, and on the west bank of the valley. Based on the inspection made at that time, the writer prepared and furnished to the Public Service Commission of West Virginia, at its request, his conclusions as to the failure of the dam. These conclusions, dated January 22d, 1914, were as follows:

1.—Complete failure occurred between Buttresses 11 and 16, involving a length of approximately 75 ft. out of a total crest length of approximately 1 075 ft. At least three additional bays, *viz.*, as far eastward as Buttress 19, were damaged so as to require practically complete rebuilding. Similarly, the faulty cut-off wall construction and certain damage done westwardly from Buttress 11 must be remedied.

2.—Failure was caused by the undermining of the over-burden or soil under the up-stream cut-off wall. The over-burden is in general clayey, but non-homogeneous. Undermining was initiated by leakage of water from the reservoir under approximately 25 ft. head through permeable over-burden not penetrated by the cut-off wall.

3.—The pressure of ice on the reservoir against the deck of the dam did not contribute to the failure.

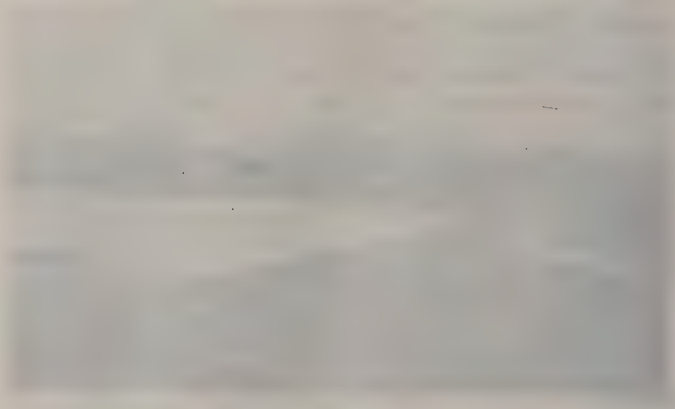
4.—The dam was of the Ambursen, hollow, reinforced concrete type. The type of dam was not in any way a cause of failure.

5.—The quality of concrete and character of construction were good.

6.—Failure occurred where the up-stream cut-off wall extended only a short depth (5 to 7 ft.) into the over-burden. Where the up-stream cut-off wall extended to a comparatively greater depth (said to be to rock), failure was checked, and the cut-off wall is still intact.

FIG. 1.—STONY RIVER DAM, AS ORIGINALLY CONSTRUCTED.

FIG. 2.—BREAK AND WASH-OUT OF STONY RIVER DAM, JANUARY 18TH, 1914.



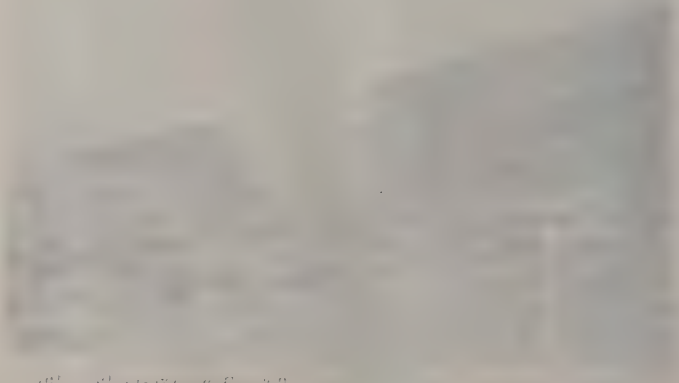


Fig. 3.—Eastern Edge of Break in Stony River Dam, January 18th, 1914.

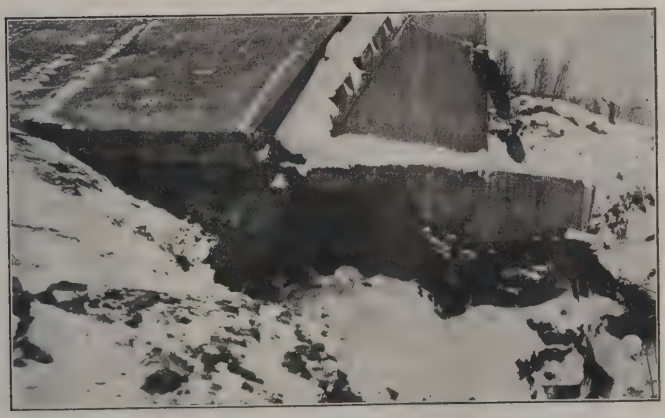
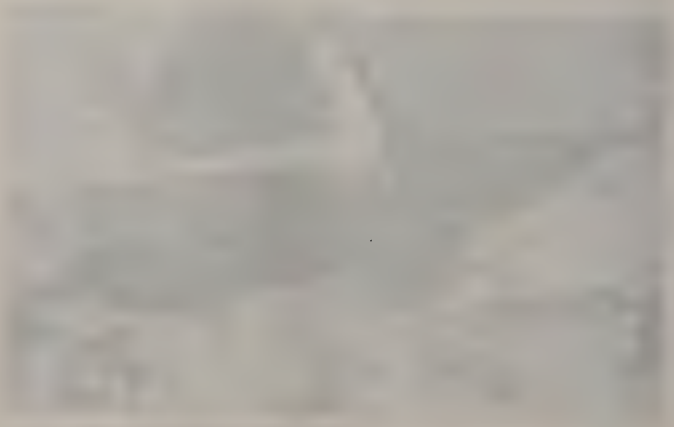


Fig. 4.—Buttress 11 of Stony River Dam, January 18th, 1914.



7.—The preliminary exploration and investigation of the dam site were not sufficiently comprehensive, and did not develop essential and all-important facts.

8.—The technical advisers consulted in connection with the foundation of the dam were not familiar with local conditions. The geological factors at the dam site were not sufficiently considered.

9.—The conditions obtaining at the Stony River dam site are such that a safe dam could be built there. Whether the portion of the dam still intact, and with its core-wall reported to extend down to rock, has sufficient factors of safety, depends on facts of which we do not have accurate knowledge. Further investigation should be made as to this question.

10.—The mechanism for operating the outlet-gates and valves is not sufficiently protected against ice formed by otherwise harmless leakage through the deck of the dam; nor is such mechanism readily accessible under all conditions.

11.—Although the dam should, and could, have been designed and constructed so as to be absolutely safe, regardless of the gate mechanism and the character of the attendance, yet, even in this instance, had the attendant been observant and resourceful, it would have been possible to open the sluice-gates in ample time to drain the reservoir sufficiently to have obviated failure of the dam.

12.—Published newspaper reports have grossly exaggerated the details of the failure and the extent of the damage to property.

In general, the foregoing summarized conclusions on the part of the writer still hold good. It may be well to point out, however, that the writer distinguishes between the "character of construction work" and the design of the structure.

The location of the break in the dam, with reference to the entire profile, is shown on Plate IX. Complete failure, as previously stated, involved a length of about 75 ft., *viz.*, between Buttresses 11 and 16, but certain additional damaged work was removed, so that the gap in the dam to be closed during the reconstruction work comprised a total length of 135 ft., *viz.*, between Buttresses 10 and 19.

In February, 1914, the writer was engaged as Consulting Engineer by the West Virginia Pulp and Paper Company to take charge of the

proposed reconstruction. He desires to emphasize the point that he was not requested to place the responsibility for the failure, nor did he attempt to do so. It is fair to note, however, that in this case, unlike most other failures, no criticism has been heard directed against the owner, that is, against the executive officers of the West Virginia Pulp and Paper Company, on the score that economy was preferred to safety. Moreover, in connection with the proposed reconstruction, the executive officers of the Company proceeded on the principle that either the dam should be reconstructed so as to be reliably safe, or no attempt should be made to utilize the damaged structure. The same attitude was manifested throughout the entire period of reconstruction.

Prior to the final decision to proceed with the actual work of reconstruction, the entire structure and foundation conditions were investigated carefully, in order to discover any features of design or construction involving insufficient margins of safety. Not until about August 1st, 1914, did the owner finally decide to reconstruct. The work was finished in May, 1915, though the storage of water in the reservoir had been resumed several months earlier.

Geological and Foundation Conditions.

The available information regarding foundation conditions was found to be so inadequate that the investigation which followed had to be made practically as complete as though there had been no dam on the site. In fact, the existence of a dam at the site necessitated additional exploration to determine the adequacy of the cut-off.

An F-1 Davis Calyx core-drill was placed on the work, and by it ten holes were sunk. The deepest hole was sunk to approximately 100 ft. below ground surface; several other holes were carried to depths of approximately 80 ft. Usually, the strata penetrated were sufficiently tight to retain in the hole the water necessary for the shot-drilling process. In other cases, however, especially on the east bank of the valley, seams were encountered in the rock, and these were of such capacity that it was difficult to keep sufficient water in the holes to allow the drilling to proceed.

Numerous test pits were sunk, the majority being immediately adjacent to, and up stream from, the original cut-off wall. Such test pits not only exposed the character of the over-burden, but also permitted an examination of the original cut-off and the efficacy of its seal into the bed-rock. Several of the test pits extended to a depth of

approximately 45 ft., and were sunk at considerable expense, because of the inflow of water and the necessity for sheeting the pits for practically their entire depth.

On Plate IX are shown the locations of the core-drill holes, the logs thereof, and the "lay" of the rock strata as deduced from the logs and other records. This plate shows, also, the profile of the original ground surface of the valley. It is apparent that the axes of the original valley and of the present valley do not coincide. By "original" valley is meant that valley which existed at the time the stream was still eroding the bed-rock and was in immediate contact with that rock. At that time the axis of the valley was approximately at the west end of the old spillway, or at about Station 5 + 50. As time went on, the stream ceased its cutting or eroding action and began to build up the floor of the valley by depositing sediment and gravel. At the sides of the valley this process was apparently aided materially by "slips" and talus from the hillsides.

During this process Stony River meandered back and forth across the valley until at present the axis of the valley is approximately at Station 4 + 40. If, in the profile shown on Plate IX, the original and the present locations of the stream-bed were to be connected by a line passing through all the intermediate locations of the stream-bed, such connecting line would have an extremely irregular or zigzag course. Water-worn boulders and gravel were found strewn through various portions of the over-burden as exposed by the exploratory work, testifying to the fact that they had been deposited by the stream in ages past.

The Stony River Valley lies in the geological formations of the Carboniferous period. The lower portion of the valley, at the dam site, lies in what is known locally as the Savage formation, one of the principal features of which is the Davis or Kittanning coal, which is of commercial importance throughout this region.

The general characteristics and slope of the formations are indicated roughly in the Piedmont folio (West Virginia) of the United States Geological Survey. The detailed geological correlations at the dam site, however, were far from simple. Variability, rather than uniformity or persistency, characterizes the Savage formation, and nowhere is this exemplified better than at the site in question. From the core-drilling data it is apparent that lentils abound, that relatively soft

strata are interbedded with hard strata, and, what is of even greater importance, that the geological horizons, which at one location offer impervious strata, show decidedly pervious strata at other locations.

The characteristic rocks are shales of a laminated or seamy character, as shown in the typical section, Fig. 6. Here and there lentils of coal and black slate are interbedded in them. At approximately the location of the break, *viz.*, in the neighborhood of Station 2 + 00, the shales were found to be so crumpled and crushed by folding near the surface as to require their excavation in the process of reconstruction.

It is not believed that this condition had any relation to the failure of the dam, but it is probably true that, once the dam had failed and the reservoir was being emptied through the break, the erosion of the bed-rock immediately down stream from the place of failure was facilitated considerably by this local surface crumpling of the shales.

At the east side of the valley (the left side of the profile in Plate IX) heavy sandstones were encountered. Evidently, at the time the valley was formed by the process of stream erosion, the sandstones caused the formation of overhanging cliffs, under which the softer strata were eroded. Frost and other natural forces, however, had their effect, even on these sandstone cliffs, with the result that they became fissured, though the boulders which were thus cracked off probably did not move far from their original location. In fact, excavations made under the original cut-off showed that in the original construction certain of these boulders or false cliffs had been mistaken for bed-rock, and that under them there was actually pervious material, such as the disintegrated coal shown in the profile immediately to the west of Station 9 + 00.

The over-burden or foundation soil at this site is mainly of a clayey nature. It is far from being homogeneous, as is evident from Fig. 7, which shows typical soil conditions at Buttress 10. In general, the soil near the surface is loamy and more pervious than that immediately overlying the bed-rock. Near the middle of the valley, however, the conditions are reversed, and there deposits of pervious materials, such as gravels, were found at considerable depths.

A catalogue of the constituent materials of the over-burden shows:

Yellow clay.....impervious, of fair bearing value.
Blue clay.....impervious, of fair bearing value.

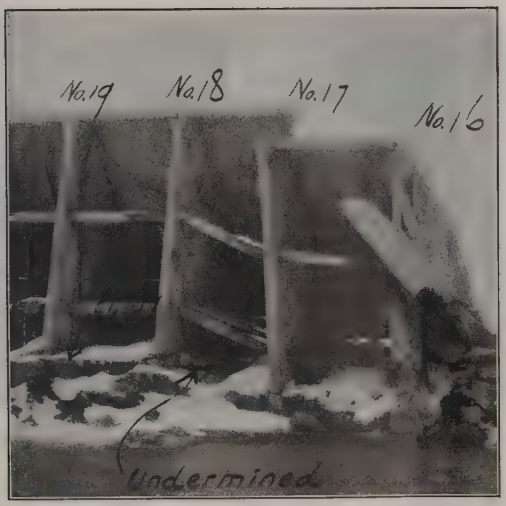


FIG. 5.—BUTTRESSES 16 TO 19, STONY RIVER DAM, MARCH 16TH, 1914.

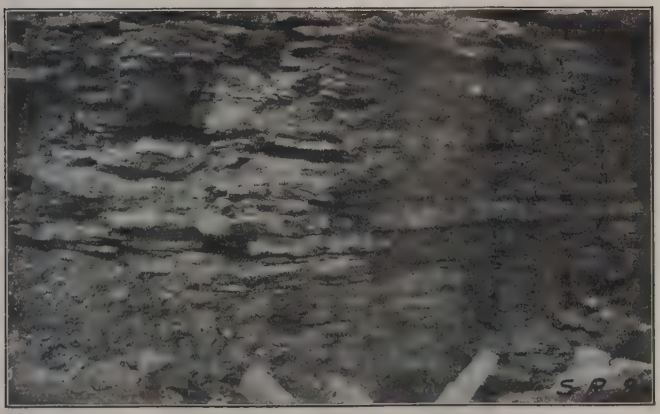
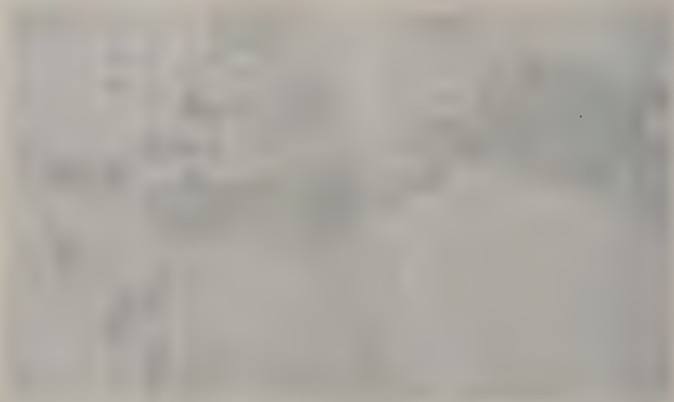


FIG. 6.—TYPICAL SECTION OF SHALE AT SITE OF STONY RIVER DAM, NEAR POINT OF FAILURE.

FIG. 7.—TYPICAL FOUNDATION SOIL CONDITIONS AT BUTTRESS 10, STONY RIVER DAM.

FIG. 8.—TEST OF FRICTIONAL RESISTANCE.



Black clay or gumbo..impervious, of fair bearing value when dry or only moist, but of poor bearing value when saturated with water.
Sand and gravel.....pervious, but of good bearing value.
Loamy sand.....in minor quantities, pervious, of poor bearing value.
Sandy clay.....fairly water-tight, of fair bearing value.
Shale fragments.....not in place, and in various stages of disintegration, mostly permeable, of fair bearing value.
Boulders.....in considerable numbers, and, of course, of excellent bearing value.

The bearing values, as referred to in the foregoing, are relative; comparison is made with the bearing values of other materials present in the over-burden, and not with those of granite or even of the shale rock on which the new buttresses, 11 to 18, inclusive, are founded. The materials occurred in no regular order.

Such foundation conditions required the most serious consideration and care in construction. In general, the designs involved in the work of reconstruction were based on the assumption that the worst conditions might exist at any bay. In view of the findings of the exploration work, it appeared logical, however, to attribute higher bearing values and greater resistance to percolation to the foundation soil of the east hillside, that is, roughly, east of Station 5 + 00, for in that portion of the valley there is more clay, and it is of a better character.

Simultaneously with the work of core-drilling and sinking test pits, the wrecked portions of the dam were removed, and the underlying foundation soil was stripped from the bed-rock, thus permitting a first-hand study of the strata on which were to be founded the buttresses which were to replace those taken out. Such excavation was actually a part of the reconstruction, inasmuch as it would have been necessary later, if it had not been made as part of the exploratory work. The investigation was completed in June, 1914, and designs and recommendations for reconstruction based on the findings were prepared immediately.

An explanation of the treatment of the more important problems of the reconstruction follows.

Spillway Provision.

Original Spillway Capacity.—A problem affecting most of the other features of design—because it determined the probable maximum head-water level, and hence affected the probable maximum stresses—was that of the spillway capacity to be provided. The original spillway provision was apparently inadequate, being only about 2 800 sec-ft. with water level at the elevation of the original crest of the bulkhead sections of the dam, *viz.*, Elevation 139 (Plate IX). In fact, it is likely that the actual safe flood discharging capacity would have been somewhat less, for the original spillway crest had not been given the most advantageous shape.

Assuming the drainage area to be 11.4 sq. miles, as shown by the "Piedmont" topographical quadrangle issued by the U. S. Geological Survey, the original spillway capacity was less than 250 sec-ft. per sq. mile of drainage area.

Pertinent Records of Flood Flow.—In general, in a given region, the smaller the drainage area the larger the maximum unit run-off and flood flow. So-called cloudbursts are limited as to area affected, but certainly the Stony River drainage area is sufficiently small to fall within the territorial limits of a cloudburst. It happens that there are two important and applicable records* of flood discharge, namely, those of the floods on:

Cane Creek, at Bakersville, N. C., May 19th-20th, 1901; drainage area, 22 sq. miles; estimated maximum discharge, 1 386 sec-ft. per sq. mile; elevation above sea level at point of measurement, approximately 2 450 ft. (U. S. G. S. datum); elevation of highest point in water-shed, approximately 5 330 ft.; and

Elkhorn Creek, at Keystone, W. Va., June 22d, 1901; drainage area, 44 sq. miles; estimated maximum discharge, 1 363 sec-ft. per sq. mile; elevation at point of measurement, approximately 1 600 ft.; elevation of highest point in water-shed, approximately 3 365 ft.

The fact that both of these extreme floods occurred in the Southern Appalachian mountain system, in which the Stony River dam is situated, and the further fact that the several drainage areas in ques-

* *Engineering News*, August 7th, 1902.

tion, including that of Stony River (with the dam at approximately Elevation 3 350 and the highest point in the water-shed at approximately Elevation 4 200), are at not widely differing altitudes, cause these two records to be exceptionally valuable for purposes of comparison. These drainage areas are apparently quite similar in their physical characteristics.

The records referred to were reported by Mr. E. W. Myers, then Engineer of the North Carolina Geological Survey, and Assistant Hydrographer of the U. S. Geological Survey. It is to be admitted that the method of measurement was probably not such as to warrant expressing the discharges to the nearest second-foot; on the other hand, an examination of the data does not warrant one in disregarding the records or assuming that the maximum flood discharges were materially less than those stated.

Although the Cane Creek flood was estimated to have been actually of slightly greater intensity than that of Elkhorn Creek, yet relatively, and for the purpose of predicting the maximum probable flood discharge from the drainage area above the Stony River dam, the Elkhorn Creek flood is of considerably greater importance, because of the fact that the drainage area at the point of measurement is twice as great as in the case of Cane Creek. For detailed data, however, one must work from the records of the Cane Creek flood, as in that case, fortunately, Mr. Myers made an estimate of the average discharge during each hour of the flood, as well as an estimate of the maximum discharge during the flood. In the graph of this flood, Fig. 12, the run-off of Cane Creek has been reduced in proportion to the drainage area, so as to be applicable to the drainage area above the Stony River Dam. Granting that the Cane Creek data are only approximate, they nevertheless appear to be well worth while adopting as a basis of study.

It is noteworthy that all available records indicate that, in the region under consideration, maximum rains, of sufficient duration to tax the equalizing capacity of the reservoir, do not occur in the early spring, and hence do not coincide with the melting of the snow. For this reason it was assumed that the greatest flood to be anticipated at the Stony River Dam would be due solely to rainfall.

Comparison with Records of Maximum Rainfall.—For practical purposes, it may be assumed that the Elkhorn and Cane Creek floods

were of equal intensity when reduced to a unit basis. The following remarks, based on the Cane Creek record, are consequently applicable also to the even more important Elkhorn Creek record.

Both these floods were undoubtedly caused by local cloudbursts which occurred after previous rains had saturated the respective drainage areas. For the purpose of comparison with rainfall records, therefore, it may be assumed that the maximum run-off per hour was approximately equal to the maximum rainfall per hour. In the case of the Cane Creek flood it is reported that, during the hour of maximum run-off, the discharge was at an average rate of 1 202 sec-ft. per sq. mile, corresponding to a rainfall at the rate of approximately 1.86 in. per hour.

Referring to Fig. 9, showing the duration of maximum intensities of rainfall in the United States, it will be noted that the maximum rate of rainfall previously derived for the Cane Creek flood falls well within the limits of the highest records shown.

Bulletin D of the U. S. Weather Bureau, the basis of the principal data of Fig. 9, includes only records obtained after the establishment of self-registering gauges in 1893, but prior to 1897. Because of the limited period which it covers, and, further, because of the limited number of precipitation stations involved, it is certain that the data of *Bulletin D* are only indicative, and not comprehensive or conclusive.

This fact is illustrated further by the record of an excessive rainfall at Raleigh, N. C., on Tuesday, July 14th, 1914, which has been plotted on Fig. 9. It should be remembered, of course, that records of excessive rainfall such as that at Raleigh are not of controlling importance in the present case, because they are not of sufficient duration to tax the equalizing capacity of the reservoir.

On Fig. 9 there has also been plotted a curve (*b*), derived from the graph shown on Fig. 12, on the assumptions that during the 90 min. of greatest run-off, in the case of the Cane Creek flood, the rainfall likewise was at its maximum, and that during such period the run-off was equal to the rainfall, variations being simultaneous. These assumptions, of course, are not strictly true, as the maximum rate of run-off could not have been equal to the maximum rate of rainfall. Necessarily, there must be some equalization, and it is evident from an inspection of the Cane Creek flood curve (*b*) of Fig.

9 that in this instance there must have been a considerable difference between the maximum rates of run-off and rainfall.

Yet, even allowing for a reasonable equalization or smoothing off of the peak of the rainfall, as compared with the peak of the run-off,

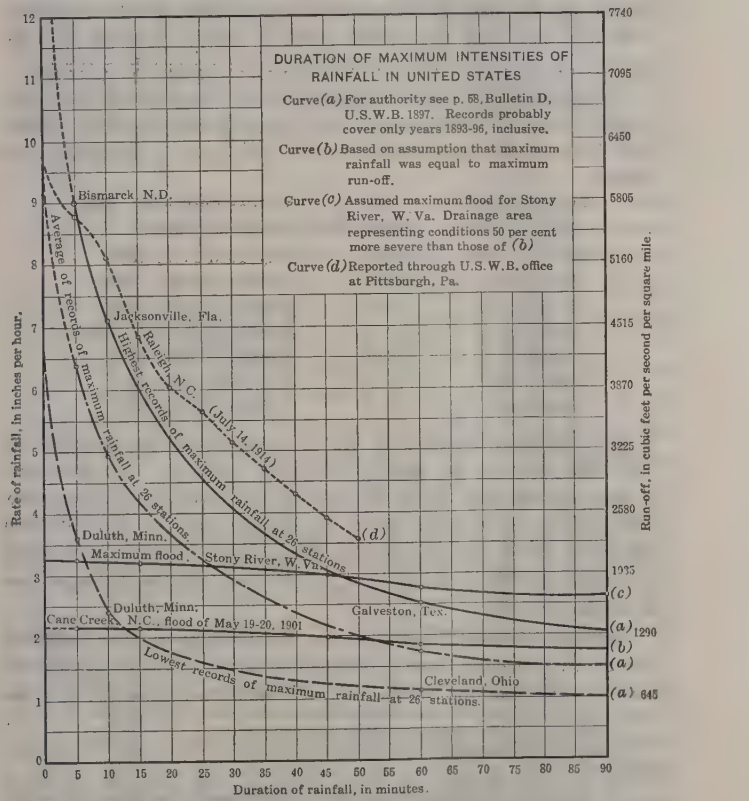

DURATION OF MAXIMUM INTENSITIES OF RAINFALL IN UNITED STATES
Curve (a) For authority see p. 58, Bulletin D, U.S.W.B. 1897. Records probably cover only years 1893-96, inclusive.
Curve (b) Based on assumption that maximum rainfall was equal to maximum run-off.
Curve (c) Assumed maximum flood for Stony River, W. Va. Drainage area representing conditions 50 per cent more severe than those of (b)
Curve (d) Reported through U.S.W.B. office at Pittsburgh, Pa.
Bismarck, N.D.
Average of records of maximum rainfall at 26 stations
Jacksonville, Fla.
Raleigh, N.C. (July 14, 1914)
Highest records of maximum rainfall at 26 stations
(d)
Duluth, Minn.
Maximum flood
Stony River, W. Va.
Duluth, Minn.
Cane Creek, N.C., flood of May 19-20, 1901
Galveston, Tex.
Lowest records of maximum rainfall at 26 stations
Cleveland, Ohio
(c)
(a) 1290
(b)
(a)
(a) 645
Rate of rainfall, in inches per hour.
Run-off, in cubic feet per second per square mile.
7740
7095
6450
5805
5160
4515
3870
3225
2580
1935
0 5 10 15 20 25 30 35 40 45 50 55 60 65 70 75 80 85 90
Duration of rainfall, in minutes.

FIG. 9.

it is apparent that the rainfall which caused the Cane Creek flood was by no means as great as may be expected in either the Cane Creek or Stony River localities. This conclusion is confirmed by a study of the total rainfall in 24 hours.

The total run-off, as shown in Fig. 12, for the 24-hour period beginning at 7 P. M., May 19th, 1901, is equivalent to a depth of 7.48 in. over the whole Cane Creek drainage area of 22 sq. miles. For the same period it is reported that records at precipitation stations reasonably near the Cane Creek water-shed show a total of 8 in. of rainfall. (The fact that these two figures check so closely indicates that the estimate of the Cane Creek flood discharge is reasonably accurate.)

As compared with this 24-hour rainfall, there were available 24-hour rainfall records at other points in the United States east of the Mississippi River which were instructive with reference to the problem, as follows:

Carlisle, Pa.....	9.35 in.	August 26th-27th, 1899.
Newton, Ala.....	10.29 "	March 22d, 1897.
Wheeler, Ohio.....	10.47 "	May 16th-17th, 1893.
Horse Cove, N. C..	11.00 "	October 3d-4th, 1898.
Morgan, Ga.....	11.52 "	March 21st-22d, 1897.
Manning, S. C....	13.22 "	August 27th-28th, 1893.
Falkland, N. C....	13.55 "	August 3d-4th, 1894.
Jewell, Md.....	14.75* "	July 26th-27th, 1897.
Sebastian, Fla....	19.08 "	October 2d-3d, 1899.

The foregoing records are by no means complete, inasmuch as they refer only to the period, 1891-1899, inclusive. On the basis of these records, and taking the geographical position and topography into account, it appeared proper to assume that the greatest flood reasonably to be provided for at Stony River would be the result of a total rainfall of about 12 in. in 24 hours on the drainage area above the dam site.†

Spillway Capacity Provided in the Reconstruction.—The foregoing considerations having made it evident that additional spillway capacity must be provided, the volume of such additional capacity, and the

* Reported actually to have occurred within 18 hours.

† Since the reconstruction, the following pertinent data have come to the attention of the writer:

(1) A run-off having a rate between 2 000 and 2 333 sec-ft. per sq. mile is reported to have occurred over a drainage area of about 1½ sq. miles near Le Roy, N. Y., in May, 1916. *Engineering News*, June 24th, 1916, p. 842.

(2) For July 8th, 1916, Professor Alfred J. Henry, U. S. Weather Bureau, reports that the area (Alabama and Georgia) covered by a 24-hour rainfall of 8 in. and more was about 4 945 sq. miles. *Monthly Weather Review*, August, 1916, p. 467.

(3) A 24-hour rainfall of 22.22 in. is reported to have occurred on July 15th-16th, 1916, at Altapass, Mitchell County, N. C. (Elevation 2 625). *Ibid*, p. 467.

(4) It is reported that rain fell at the rate of 5.21 in. per hour for 25 min. at Mobile, Ala., on July 8th, 1916. *Ibid*, p. 468.

means of providing it, remained to be determined. No attempt will be made in this paper to discuss the alternatives considered for increasing the original spillway capacity of the dam, for such alternatives differed essentially only as to the extent to which the several available factors were utilized. Instead, the conditions caused by the maximum flood within the limits of reason will be applied to the means finally adopted for increasing the spillway capacity, in order to test the adequacy of these means. The adopted spillway provision, which will be understood by reference to Plate IX, was obtained by the following means:

1.—Reconstructing that portion of the original bulkhead section of the dam between Buttresses 10 and 19 to form a second spillway. The transverse cross-section of this new spillway is shown in Plate X. It was provided with a suitable reinforced concrete channel mat extending about 85 ft. down stream from the bucket of the spillway apron.

2.—Adding 3 ft. 6 in. to the height of the original bulkhead portions of the dam outside of the spillways, *viz.*, east of the old spillway and west of the new spillway. This was accomplished by adding a parapet, of the form shown in outline in Plate XII, with its top at Elevation 142.5.

3.—Increasing the height of that portion of the original bulkhead section of the dam between Buttresses 19 and *A* (*viz.*, between the old and new spillways) by the addition of a parapet 1 ft. high. This forms an intermediate spillway with its crest at Elevation 140, as shown in the typical sections of Plate XII.

4.—Utilizing the horizontal upper member of the new anchoring wall at the toe of the dam, between the old and new spillways, to form a reinforced concrete mat or "tumbling hearth" to receive the impact of the sheet of water falling well-nigh vertically from the intermediate spillway. This horizontal mat, together with an inclined, reinforced concrete, protection mat (section *B-B* of Plate XII), forms a channel sloping downward from the ends of the intermediate spillway section toward, and discharging through, a new outlet channel which extends down stream from Gate-bays 30 and 31.

5.—Improving the shape of the crest of the original or "old" spillway. This warranted the assumption of a somewhat higher coefficient of discharge, as applied in the Francis formula.

Under the original conditions, disregarding wave action, overtopping of the dam would have occurred when the water level in the reservoir had reached Elevation 139; whereas, under the conditions obtaining after reconstruction, overtopping cannot occur until the water level reaches Elevation 142.5. It is probable that overtopping due solely to wave action would be of minor consequence. However, as will presently be shown, the water level due to the greatest flood reasonably to be provided for would probably leave a margin of about 6 in. below Elevation 142.5.

Maximum Flood Within Limits of Reason.—For the purpose of studying the effect of such a flood, in the case of the Stony River Dam and Reservoir, let it be assumed that it would have a graph similar to that shown in Fig. 13. This graph was derived by increasing the rates of run-off shown in Fig. 12 by 50%, thus corresponding to the ratio of the estimated maximum 24-hour precipitation to the 8-in. precipitation reported to have occurred within a like period in the case of the Cane Creek flood. It is true, of course, that no two rain storms or floods are exactly alike; yet, for the purpose in mind, the foregoing assumption appeared to afford a reasonable basis for design.

For purposes of comparison, there has been plotted on Fig. 9 a curve (*c*) showing the rainfall for the critical 90-min. period of such assumed greatest possible flood discharge, the curve being based, as before, on the assumption (not strictly true) that the rate of rainfall would be equal to the rate of run-off. Even allowing for the evident equalization, or smoothing-off of the peak of the rainfall, it is apparent that a rainfall causing a flood such as that represented by the curve (*c*) would not equal certain actual records of rainfall.

Equalization Effect of Reservoir.—In studying the results of such floods as those plotted on Figs. 12 and 13, when applied to the Stony River drainage area and spillway provision, it may reasonably be assumed that in each instance there would be, initially, one point of time when, for all practical purposes, a state of equilibrium would exist.

At such a time the total run-off from the drainage area and the discharge over the spillway would be equal. Thereafter, as the run-off increased, a certain quantity of water would be stored over the whole reservoir area for each foot of rise in water level. The extent of such equalization or absorption would depend on the spillway capacity and the reservoir area.

The capacity of the reservoir above the elevation of the lower spillway crests is shown on Fig. 10. On Fig. 11 is plotted the spillway capacity (after reconstruction), derived from the data and assumptions there shown.

On the basis of these diagrams, the equalization effect of the reservoir has been studied with reference to a number of flood conditions, as follows:

I.—Before considering the greatest flood reasonably to be anticipated, it appears proper to study the equalization effect of the reservoir on a flood exactly similar, and equal in intensity, to that reported for Cane Creek on May 19th-20th, 1901. The result of such study is shown in Fig. 12. The spillway crests are assumed to be without flash-boards.

The method of deriving the curve showing the discharge over the spillways is as follows: Referring to Fig. 12, it is assumed that at 7 A. M. the total run-off from the drainage area and the discharge over the spillways are equal. The graph of the flood then shows that at this time the run-off would be approximately 100 cu. ft. per sec., which (from Fig. 11) would require the reservoir water level to be at Elevation 136.2. The effect of the storage capacity of the reservoir at increasing elevations of head-water is then determined at arbitrary intervals, depending on the refinement desired. Selecting a 0.3-ft. rise in water level as the first interval, it is found from the reservoir capacity curve of Fig. 10 that for this interval the reservoir will absorb approximately 39 000 000 gal. This quantity, when converted into the proper units and scale, is represented in Fig. 12 by the area, *a-b-c, viz.*: The point, *b*, of course, must have an ordinate representing the spillway discharge at Elevation 136.5, approximately 365 cu. ft. per sec. (from Fig. 11). The position of the line, *b-c*, is then determined by trial, the required area, *a-b-c*, having been determined as above stated.

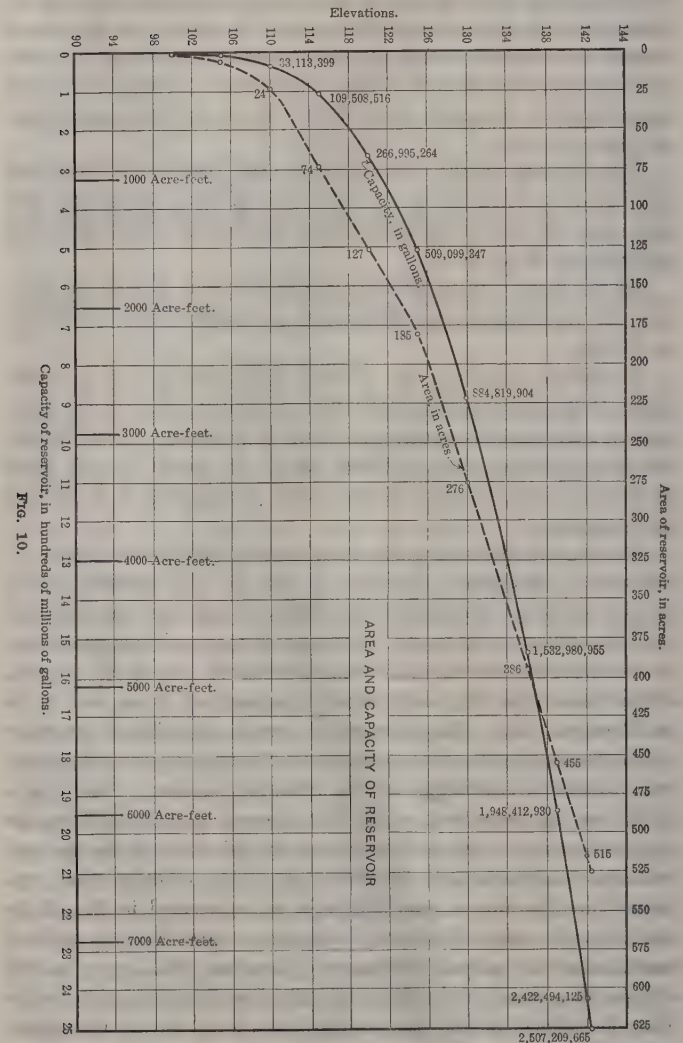

Elevations.
90 94 98 102 106 110 114 118 122 126 130 134 138 142 144
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25
0 25 50 75 100 125 150 175 200 225 250 275 300 325 350 375 400 425 450 475 500 525 550 575 600 625
Capacity of reservoir, in hundreds of millions of gallons.
Area of reservoir, in acres.
1000 Acre-feet.
2000 Acre-feet.
3000 Acre-feet.
4000 Acre-feet.
5000 Acre-feet.
6000 Acre-feet.
7000 Acre-feet.
Capacity, in gallons.
Area, in acres.
AREA AND CAPACITY OF RESERVOIR
33,113,399
24
109,508,516
74
266,995,264
127
509,099,347
185
884,819,904
276
1,532,980,955
386
1,948,412,930
455
515
2,422,494,125
2,507,209,665

FIG. 10.

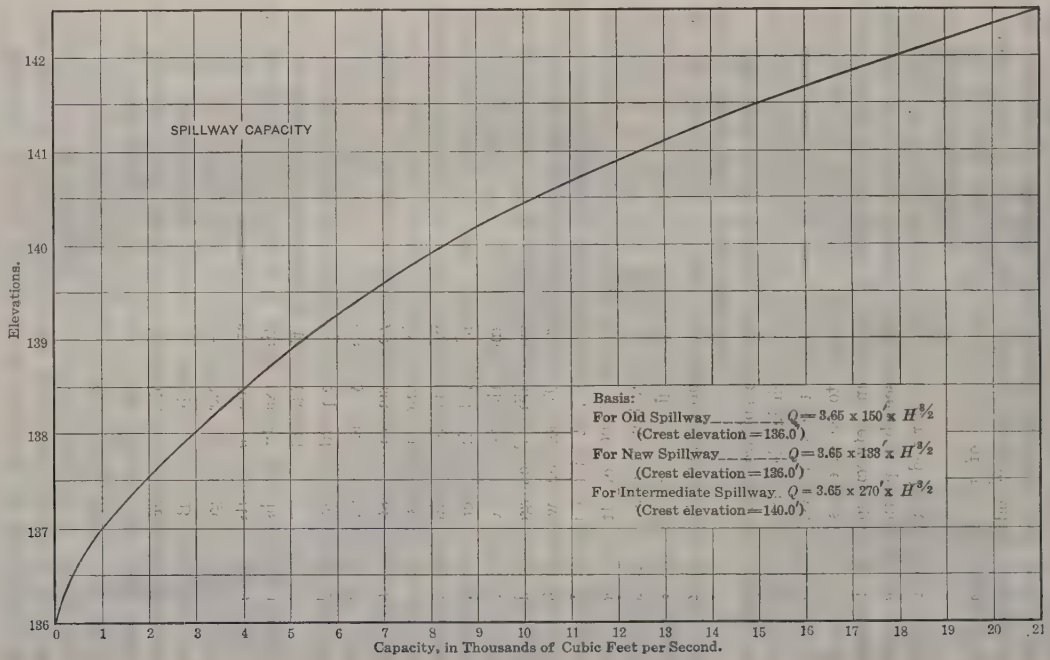

SPILLWAY CAPACITY
Elevations.
142
141
140
139
138
137
136
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21
Capacity, in Thousands of Cubic Feet per Second.
Basis:
For Old Spillway_____ $Q = 3.65 \times 150' \times H^{3/2}$
(Crest elevation = 136.0')
For New Spillway_____ $Q = 3.65 \times 133' \times H^{3/2}$
(Crest elevation = 136.0')
For Intermediate Spillway $Q = 3.65 \times 270' \times H^{3/2}$
(Crest elevation = 140.0')

FIG. 11.

Similarly, a 0.5-ft. rise in water level may be selected for the next interval. When the head-water has reached Elevation 137.0, the spillways will be discharging approximately 1 030 cu. ft. per sec. (Fig. 11), thus determining the ordinate of the point, *e*, of Fig. 12. Meanwhile, approximately 66 000 000 gal. (Fig. 10) will have been stored in the upper 0.5-ft. depth of the reservoir, thus determining the area, *b-c-d-e*, and hence the position of the line, *e-d*. By repeating this process, the curve of Fig. 12 representing "Discharge over spillways" is defined. The last interval—that between the points, *f* and *g*—must be determined by trial so as to utilize exactly the absorption capacity of the reservoir corresponding to that interval.

It is evident that the ordinate of the point, *g*, represents the spillway capacity necessary to pass safely the particular flood under investigation. In point of time, *g* represents the crest of the discharge over the spillways, though, in this instance, the crest of the spillway discharge does not occur until approximately 35 min. after the flood itself has reached its peak. From Fig. 12 it is seen that the maximum head-water level attained under the assumed conditions would be at Elevation 140.36. The corresponding discharge over the spillways would be approximately 9 600 cu. ft. per sec., whereas the available spillway capacity, with head-water just at the top of the bulkhead section parapets, is approximately 21 000 cu. ft. per sec. This appears to be more than ample margin, but it must be remembered that the graph of Fig. 12 represents a flood which actually occurred on the Cane Creek drainage area—approximately twice as great as that of Stony River at the dam site.

The crest of the discharge over the spillways having passed, the reservoir begins to yield the water which has been temporarily stored in that portion of the reservoir above the spillways. Therefore the discharge over the spillways continues to be in excess of the run-off, until finally the condition of equilibrium between spillway discharge and run-off is again reached. The descending portion of the curve, showing "Discharge over spillways," was derived by the same method as the ascending portion.

II.—Turning now to the consideration of the maximum flood within limits of reason, Fig. 13 shows the equalization capacity of the reservoir as affecting a flood exactly similar to, but with a unit discharge 50% greater than, the 1901 Cane Creek flood. The required

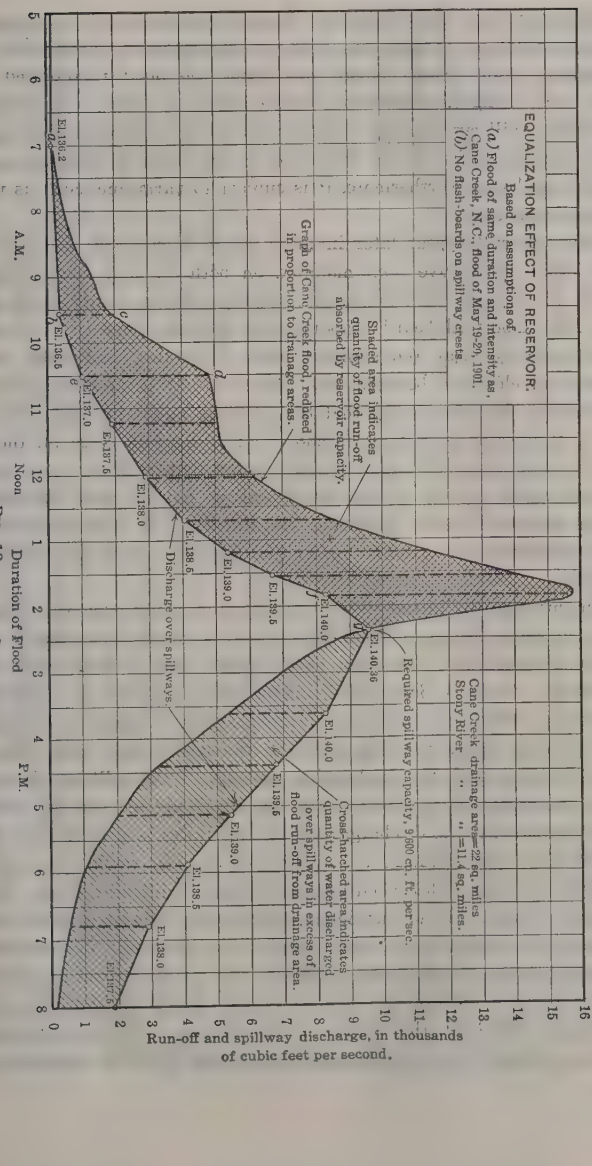

EQUALIZATION EFFECT OF RESERVOIR.
Based on assumptions of
(a.) Flood of same duration and intensity as Cane Creek, N. C., flood of May 19-20, 1901.
(b.) No flash boards on spillway crests.
Cane Creek drainage area=22 sq. miles.
Stony River " =11.4 sq. miles.
Required spillway capacity, 9,600 cu. ft. per sec.
Shaded area indicates quantity of flood run-off absorbed by reservoir capacity.
Graph of Cane Creek flood, reduced in proportion to drainage areas.
Cross-hatched area indicates quantity of water discharged over spillways in excess of flood run-off from drainage area.
Discharge over spillways.
El. 140.36
El. 140.0
El. 139.6
El. 139.0
El. 138.6
El. 138.0
El. 137.6
El. 137.0
El. 136.5
El. 136.2
El. 139.0
El. 138.5
El. 138.0
El. 137.8
a
b
c
d
e
A.M.
Noon
P.M.
Duration of Flood
Run-off and spillway discharge, in thousands of cubic feet per second.

Fig. 12.

spillway capacity, derived by the method just described, is approximately 15 600 cu. ft. per sec., with the corresponding head-water at Elevation 141.6. Even though we accept the flood represented in Fig. 13 as being the maximum within limits of reason, the margin, or surplus spillway capacity, does not appear to be unreasonably large when it is remembered that this flood is essentially an arbitrary conception, and that, furthermore, it is subject to variations such as will now be considered.

III.—As will appear later in more detail, flash-boards about 3 ft. high (Plate IX) have been mounted on both old and new spillway crests, the tops of the flash-boards, therefore, being nearly at Elevation 139. The flash-board supports have been designed so as to fail and absolutely clear the spillway crest when the head-water level reaches an elevation between 140 and 140.5. It is proper, however, to consider the contingency that the supports might not fail with so low a head-water level as is intended. The diagram, Fig. 14, therefore has been prepared under the assumption that, with the same flood as that treated in Fig. 13, the flash-boards do not fail until the water in the reservoir reaches Elevation 141, namely, 5 ft. higher than the main spillway crests and 1 ft. higher than the crest of the intermediate spillway.

In this case the initial state of equilibrium would exist with head-water at about Elevation 139.3, that is, 0.3 ft. above the assumed top of the flash-boards. The effect of the initial rise of head-water (to Elevation 141.0) is determined in the same manner as described previously. With the breaking of the flash-board supports, however, and the consequent sweeping away of the flash-boards when Elevation 141 is reached, the discharging capacity of the spillways is suddenly increased—as is evident from the diagram—to such an extent that, temporarily, the discharge over the spillways is greater than the run-off from the drainage area. This, of course, would not necessarily be the case with a flood having a graph of different shape. The position of the point, *c*, was determined by trial, the area, *a-b-c*, representing the quantity of water stored in the reservoir in the depth interval between points, *c* and *a*, because this quantity must be yielded by the reservoir before the head-water can recede to approximately Elevation 140.52 at the point, *c*. Actually, *a-c* would undoubtedly not be a straight line; a more accurate location would be obtained by

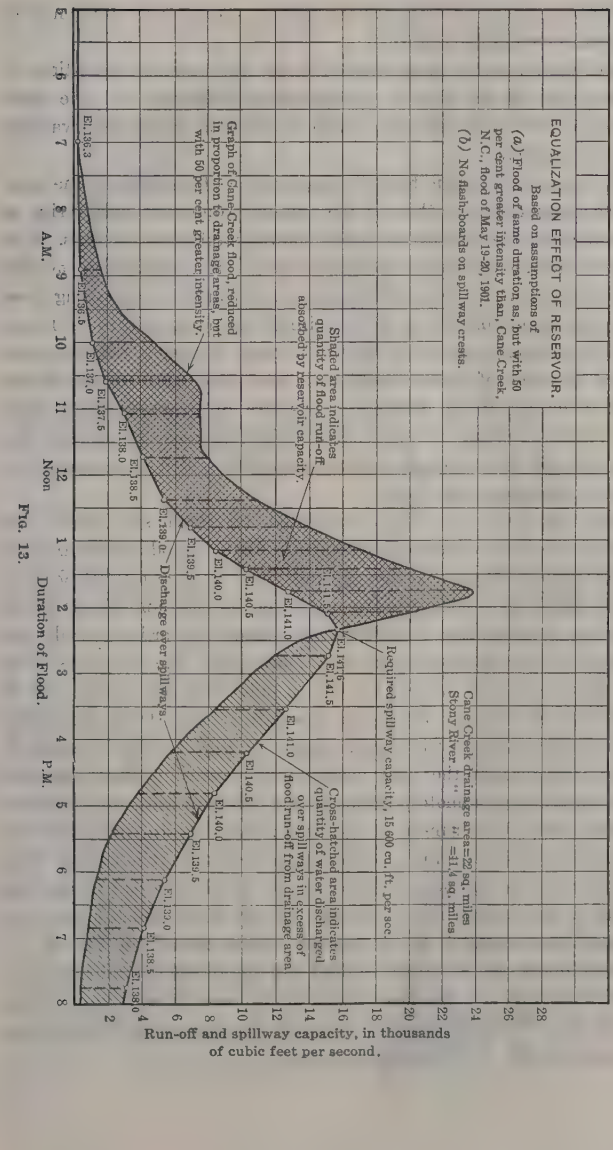

EQUALIZATION EFFECT OF RESERVOIR.
Based on assumptions of
(a) Flood of same duration as, but with 50 per cent greater intensity than, Cane Creek, N. C., flood of May 19-20, 1901.
(b) No flash-boards on spillway crests.
Cane Creek drainage area=29 sq. miles
Stony River " " =11.4 sq. miles
Required spillway capacity, 15 600 cu. ft. per sec.
Shaded area indicates quantity of flood run-off absorbed by reservoir capacity.
Cross-hatched area indicates quantity of water discharged over spillways in excess of flood run-off from drainage area.
Graph of Cane Creek flood, reduced in proportion to drainage areas, but with 50 per cent greater intensity.
Discharge over spillways
El. 141.5
El. 141.0
El. 140.5
El. 140.0
El. 139.5
El. 139.0
El. 138.5
El. 138.0
El. 137.5
El. 137.0
El. 136.5
El. 136.3
El. 139.0
El. 138.5
El. 138.0
A.M.
Noon
P.M.
Duration of Flood.
Run-off and spillway capacity, in thousands of cubic feet per second.

Fig. 13.

the determination of intermediate points, though the difference in final result would be slight.

The highest water level reached under the assumed conditions is at Elevation 141.92, with a corresponding discharge over the spillways of 17 500 cu. ft. per sec., which is still well within the available spillway capacity.

IV.—Again, it is true that the equalization of the peak of the rainfall, as compared with that of the run-off, would not necessarily be as great as in the case of the floods shown graphically on Figs. 12 to 14; and, though the writer does not consider it reasonable to assume that the Stony River Dam will be subject to conditions resulting from more than approximately a 12-in. rainfall occurring within 24 hours, yet, thus far, only one of an infinite number of graphs has been considered, any one of which would yield the same total run-off during the 24-hour period in question. The use of graphs having other shapes (but enclosing the same area) might increase or decrease the proportion of the available spillway capacity which is necessary to pass the corresponding flood.

Only one variation will be considered, for which see Fig. 15. It appears to be not reasonably conceivable that a flood resulting from a 12-in. precipitation in 24 hours (and yielding almost as much total run-off within the same period) could reach its peak and recede therefrom more quickly than that there represented. In order to make the severity of the assumed conditions more clear, the ordinates have been shown in terms both of run-off, in cubic feet per second, and of rainfall, in inches per hour. On the latter basis, these conditions may be compared with the records shown on Fig. 9. For instance, if, taking into account the time necessary for water to reach the dam site from the most remote portions of the drainage area, it is assumed that the peak of the run-off at the dam is equal to the greatest average rate of rainfall during any 45-min. period, then it is evident that, in order to cause the flood in question, it would have had to rain for 45 min. at an average rate of about 4 in. per hour. Such a rate corresponds approximately to the highest record plotted on Fig. 9.

Even under the severe, if indeed not improbable, conditions represented in Fig. 15, the head-water would reach a maximum elevation of only 142.05, with a corresponding discharge over the spillways of approximately 18 300 cu. ft. per sec.

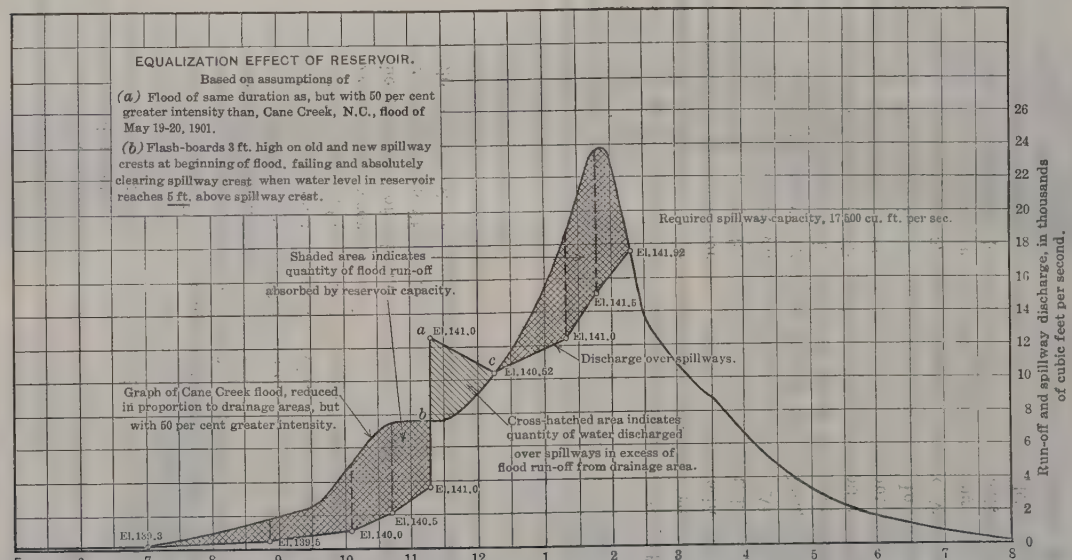

EQUALIZATION EFFECT OF RESERVOIR.
Based on assumptions of
(a) Flood of same duration as, but with 50 per cent greater intensity than, Cane Creek, N.C., flood of May 19-20, 1901.
(b) Flash-boards 3 ft. high on old and new spillway crests at beginning of flood, failing and absolutely clearing spillway crest when water level in reservoir reaches 5 ft. above spillway crest.
Required spillway capacity, 17,500 cu. ft. per sec.
El. 141.92
El. 141.5
El. 141.0
Shaded area indicates quantity of flood run-off absorbed by reservoir capacity.
a El. 141.0
c El. 140.52
Discharge over spillways.
Graph of Cane Creek flood, reduced in proportion to drainage areas, but with 50 per cent greater intensity.
b
Cross-hatched area indicates quantity of water discharged over spillways in excess of flood run-off from drainage area.
El. 141.0
El. 140.5
El. 140.0
El. 139.5
El. 139.3
5 6 7 8 9 10 11 12 1 2 3 4 5 6 7 8
A.M.
Noon
P.M.
Duration of Flood.
0 2 4 6 8 10 12 14 16 18 20 22 24 26
Run-off and spillway discharge, in thousands of cubic feet per second.

FIG. 14.

Conclusions as to Spillway Provision.—After such studies as the foregoing, the writer came to the conclusions that:

1.—The spillway capacity provided in the reconstruction of Stony River Dam is adequate.

2.—Such spillway capacity is not excessive, bearing in mind that:

(*a*) There is no warrant for assuming that during the life of the dam the Stony River drainage area will escape rainfall and resulting run-off such as have occurred in other regions which are comparable;

(*b*) The number of precipitation stations and records, especially in the Appalachian Mountain region, is not sufficient to warrant the assumption that the worst possibilities have been observed and recorded; and

(*c*) The provision of adequate spillway capacity in the reconstruction of the Stony River Dam did not involve unreasonable cost, though, of course, such spillway capacity could have been provided more cheaply in the original construction than in the reconstruction.

3.—It is reasonable to assume that the water level in the reservoir will not be higher than Elevation 142.25, under the most severe conditions within the limits of reason. This elevation of head-water, therefore, was adopted for design purposes.

4.—Granting the necessity for making provision to pass safely the greatest flood reasonably to be anticipated, it is also true that such a flood would be far from normal. Hence, in considering the "normal maximum load" to which the dam will be subjected, it appeared proper to assume the head-water at Elevation 140.5. At this elevation the flash-board supports (on the old and new spillways) are reasonably certain to have failed. The flash-boards and supports having thus been swept from the main spillway crests, the spillway capacity would be approximately 10 000 cu. ft. per sec., without taking into consideration any equalizing effect of the reservoir. In the case of the original structure, the spillway capacity likewise was approximately 10 000 cu. ft. per sec. at Elevation 140.5, it being noted, however, that such head-water elevation could not have existed without causing the original bulkhead sections (with crest at Elevation 139) to be overtopped. The corresponding unit spillway capacity is about 875 cu. ft. per sec. per sq. mile of drainage area.

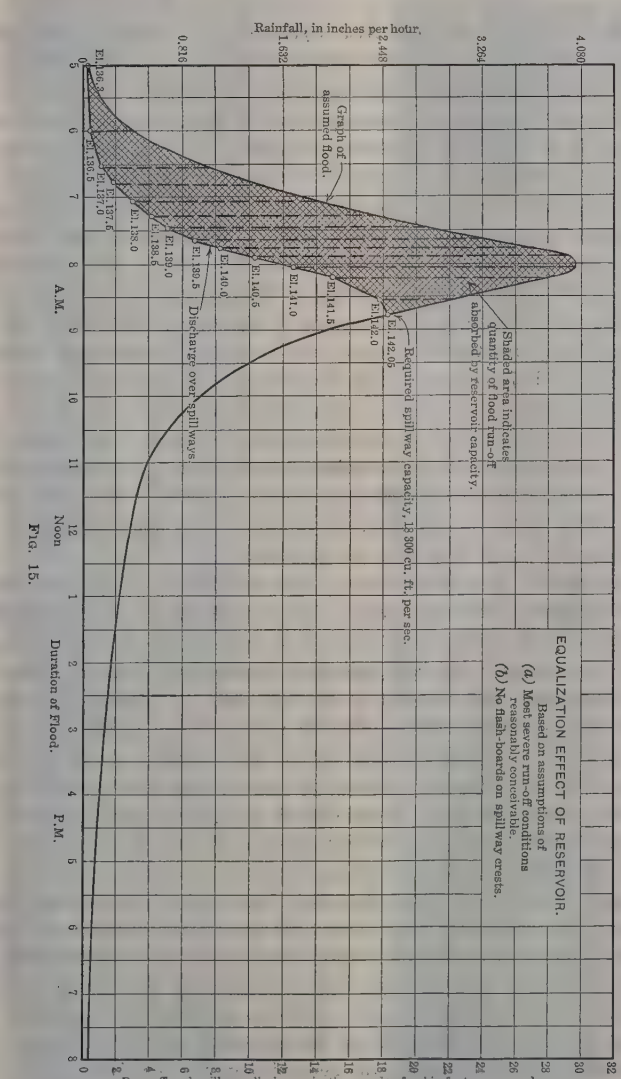

Rainfall, in inches per hour.
4.090
3.264
2.448
1.632
0.816
0
EQUALIZATION EFFECT OF RESERVOIR.
Based on assumptions of
(a) Most severe run-off conditions reasonably conceivable.
(b) No flash-boards on spillway crests.
Shaded area indicates quantity of flood run-off absorbed by reservoir capacity.
Required spillway capacity, 13 300 cu. ft. per sec.
El. 142.0
El. 142.05
El. 141.5
El. 141.0
El. 140.5
El. 140.0
El. 139.5
Discharge over spillways.
Graph of assumed flood.
El. 139.0
El. 138.5
El. 138.0
El. 137.6
El. 137.0
El. 136.5
El. 136.0
El. 135.5
A.M.
Noon
P.M.
Duration of Flood.
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
Run-off and spillway discharge in thousands of cubic feet per second.

Fig. 15.

Type of Structure for New Spillway.—It appeared logical to reconstruct the new spillway section of the dam according to the Ambursen, hollow, reinforced concrete type, provided there was no serious and valid counter consideration. Probably the most serious questions in regard to the Ambursen type of dam are those dealing with the protection of the embedded reinforcing steel from corrosion, and the extent to which the dimensions of the reinforced concrete members may properly be pared down to theoretical limits. Both these questions may be answered satisfactorily, so the writer thinks, by a sufficiently conservative design of the members of the structure.

On the other hand, the advantage of economy lay with the hollow, rather than with the solid, type, a by no means unimportant factor being the relatively high cost of the materials composing the concrete. For equal margins of safety, a solid dam would have involved somewhat greater quantities of concrete, although probably not of excavation. Furthermore, the construction of a solid section at the new spillway would have caused difficulties, costly to meet, at Buttresses 10 and 19 by reason of subjecting those buttresses to lateral hydrostatic pressure for which they were not designed.

In view of its secluded location, the appearance of the structure is of relatively minor importance. Yet even this consideration argued in favor of the hollow type, because, at least on the up-stream side, the solid section would have caused an incongruous break in outline.

Connecting with the foot of the new spillway apron, a channel mat was constructed which gradually narrowed to a width of 60 ft. at a distance averaging about 85 ft. down stream, as shown in Plate XI. The minimum depth of the channel mat varies from 30 in. on clay (24 in. on rock) at the foot of the spillway apron to 18 in. at the down-stream end of the mat, where a shallow cut-off wall seals the mat into bed-rock. Reinforced concrete retaining walls (Figs. 16 and 17) were built along the sides of the channel mat, to support the hillside on the west and the new fill between the spillways on the east. Excavation of the new spillway channel was not completed down stream from the channel mat. Only a small ditch was dug to drain water from the channel mat into the river, and the first flood practically completed the remainder of the excavation necessary to furnish sufficient connection between the new spillway and the river.

Fig. 16.—Eastern Part, Showing Old Spillway.

Fig. 17.—Western Part, Showing New Spillway.

Resistance to Sliding.

To the writer it appears that, in the case of many masonry dams, solid and hollow, the importance of safeguarding the structure against failure by sliding has been under-rated. It is not unlikely that there have been more failures of masonry dams by sliding than in any other manner. The character of the underlying foundation material is necessarily the primary factor controlling the stability of a given dam in regard to sliding. The construction of a dam to be founded on granite, for instance, offers no difficulties in this respect. But, in view of the foundation material underlying the Stony River site, the necessity of making the dam safe against sliding presented the most important problem of the reconstruction.

Uplift pressure, too, is of great importance as affecting the stability of a dam against sliding. It acts essentially to diminish the weight of the structure. The effect of a given uplift pressure, however, depends largely on the coefficient of frictional resistance of the foundation material at the plane along which sliding may be impending; that is, the smaller the coefficient of frictional resistance, the less the net effect of the uplift pressure.

Frictional Resistance.—The term "frictional resistance", as used in this paper, is intended to express both the effect of roughness and the effect of adhesion between any two bodies (*e. g.*, of foundation material) at the surfaces along which sliding is taking place. It is not intended to express the effect of cohesion within a body; such cohesion is an additional factor resisting sliding, and, in so far as resistance to sliding is concerned, must be overcome by shearing the body in question.* The distinction here made between adhesion and cohesion does not exist when the material under consideration is viscous, but, in the writer's opinion, it exists definitely when the material in question is solidified.

As to the actual coefficients of frictional resistance, there is a woeful dearth of data. Attention is called first to Table 1 which shows the assumptions on which the reconstruction design was based. The

* Since the reconstruction was completed, the writer has considered carefully the paper by William Cain, M. Am. Soc. C. E., entitled "Cohesion in Earth: The Need for Comprehensive Experimentation to Determine the Coefficients of Cohesion", *Transactions*, Am. Soc. C. E., Vol. LXXX, p. 1315, and the discussions thereon, but has been unable to agree that data, sufficient in number and reliability, are available to warrant the use, especially in the present case, of the analysis set forth in that paper.

TABLE 1.—Coefficients of Frictional Resistance Forming the Basis of Design in the Reconstruction of Stony River Dam.

Character of foundation material.	Value assumed for design.	Probable minimum value.	Ratio.
Clayey overburden—wet..... Applying to footings of the original sections of dam and to probable "planes of least resistance" in clayey foundation soil.	0.33	0.25	0.75
Shale with clay seams..... Applying to shale near surface of bed-rock; hence including shale immediately underlying new footings at Buttresses 10 to 19, inclusive (new spillway section.)	0.40	0.25	0.625
Hard shale, not disintegrated..... Applying to shale at probable "planes of least resistance" at Buttresses 10 to 19, inclusive (new spillway section.)	0.50	0.40	0.80

"planes of least resistance" referred to in that table are explained later.

The pertinent data considered in adopting these assumptions follow.

Frictional Resistance of Clayey Soil.—In the writer's opinion, the safe coefficient of frictional resistance for the clayey foundation soil at the dam site when wet is between 0.3 and 0.4. When such soil is dry or merely moist, the frictional resistance, of course, is greater than when it is thoroughly wet; but it is not safe to assume that the foundation soil will always be fairly dry. It is true that the leakage from the reservoir into the foundation soil has, at this writing, become, and in all probability will continue to be, a relatively minor factor. However, ground-water in the soil cannot be disposed of, especially inasmuch as the dam site receives the ground-water drainage of the adjoining hillsides. The new drainage system of the dam cannot cause the foundation soil to become dry. It can merely prevent the accumulation of serious uplift pressure.

The results of various tests of frictional resistance are of record. Unfortunately, they are not numerous; nor are the test conditions sufficiently standardized, nor the character of the material under tests sufficiently well defined, to allow satisfactory detailed comparison with each other or with the conditions to which it is attempted to apply their results. Moreover, it is the rule, rather than the exception, that, as in the case in point, the foundation material is heterogeneous, thus increasing the difficulty of drawing comparisons with previous test

results and making more necessary the use of judgment in selecting coefficients for design or other purposes.

Among the recorded data are the experiments of General Morin, as referred to by Rankine,* which indicate a coefficient of frictional resistance of 0.31 for wet clay on wet clay, as tested by him. Tests of the foundation soil in question, however, are more pertinent, and such tests were made on several kinds of soil by Mr. D. N. Showalter, Resident Engineer, under the direction of the writer. The results of these tests are given in Table 2.

In the ordinary case, these tests were made by tamping a given kind of soil into a box having an open top, the box being a cube of about 12 in., then capping the boxful of soil with a solid piece of the same material, so that it protruded above the edge of the box, and turning the box upside down. The protruding material, after having been given an approximately horizontal surface and made smooth, was utilized as the moving element in determining the frictional resistance on a prepared, smooth, horizontal bed of the same kind of material in place. The moving element was drawn along by hand over the bed, the required pull being measured by a spring balance. Ordinarily, more than $\frac{1}{3}$ sq. ft. of the material under test was actually in contact. The method of making the tests is illustrated by Figs. 8 and 18.

In the case of the tests made in cold weather, the clay bed had been scraped previously so as to remove all frozen material and any "mud" formed by melting snow. When not under test, the surface or bed was protected by a covering of tar-paper. In the last series of tests, in May, 1915, a galvanized-iron pail was used instead of a wooden box. The result was to make these experiments appear even more crude than was really the case; but, inasmuch as the weight of the moving element and the required pull were not less readily determinable, such experiments are not believed to be the less worthy of consideration.

Despite the absence of laboratory refinements, the results obtained are in all probability reasonably indicative of the actual frictional resistance, as previously defined. Among the results of any single group of experiments, the maximum variation from the mean was 30% of the mean, and the average of such maximum variation for all

* "A Manual of Applied Mechanics", p. 211.

TABLE 2.—Results of Tests of Frictional Resistance of Foundation Soil at Stony River Dam Site.

Experiment No.	Character of soil under test.	Condition of surfaces in contact.	Temperature range, in degrees Fahrenheit.	Load, in pounds.	Pull (while in motion), in pounds.	Coefficient of frictional resistance.
1....	White clay, fine-grained.....	Wet.	35 to 41	160	64	0.40
2....	do.	do.	do.	160	67	0.42
3....	do.	do.	do.	284	80	0.28
4....	do.	do.	do.	284	72	0.25
5....	do.	do.	do.	160	56	0.34
					Average	0.34
6....	White clay, fine-grained.....	Moist.	35 to 41	160	75	0.47
7....	do.	do.	do.	160	72	0.45
8....	do.	do.	do.	284	118	0.42
9....	do.	do.	do.	220	78	0.35
10....	do.	do.	do.	220	85	0.39
					Average	0.42
11....	Yellow clay, containing some grit.	Fairly wet.	20 to 25	149	120	0.80
12....	do.	do.	Soil was thawed by a fire near-by.	149	120	0.80
13....	do.	do.		149	106	0.71
14....	do.	do.		149	114	0.76
15....	do.	do.		189	110	0.58
16....	do.	do.		189	99	0.52
17....	do.	do.		249	120	0.48
18....	do.	do.	do.	289	224	0.77
19....	do.	do.	do.	387	273	0.71
20....	do.	do.	do.	387	275	0.71
					Average	0.68
21....	Black gumbo.....	Moist.	50 to 66	53	41	0.77
22....	do.	do.	do.	81	70	0.86
					Average	0.82
23....	Black gumbo.....	Wet.	50 to 66	50	25	0.50
24....	do.	do.	do.	79	70	0.89
25....	do.	do.	do.	79	58	0.73
					Average	0.71
26....	Black loam.....	Moist.	42 to 56	57	44	0.77
27....	do.	do.	do.	89	60	0.67
28....	do.	do.	do.	113	89	0.79
					Average	0.74
29....	Black loam	Wet.	42 to 56	57	38	0.67
30....	do.	do.	do.	89	69	0.78
					Average	0.72
31....	Yellow clay.....	Moist.	42 to 56	46.5	39	0.84
32....	do.	do.	do.	79.5	67	0.84
33....	do.	do.	do.	111.5	90	0.81
					Average	0.83
34....	Yellow clay.....	Very wet.	42 to 56	79.5	42	0.53
35....	do.	do.	do.	111.5	52	0.47
					Average	0.50

FIG. 18.—TEST OF FRICTIONAL RESISTANCE.

FIG. 19.—DEEP DRAINS.

groups was about 15 per cent. The series of tests was not sufficiently extensive to warrant any conclusions as to variations in the coefficient of frictional resistance with variations in the load or in the velocity of the moving element.

The pull or force required to start the load in motion has not been shown in Table 2, for it is believed that, in order to make a reasonably close determination of the starting force, greater refinements are necessary than could be applied in the field experiments in question. It should be noted, however, that, in view of the method of testing used, the shearing resistance (due to cohesion) of the material under test was not brought into play.

The average of the results of all the experiments shows a coefficient of frictional resistance, including adhesion, equal to about 0.61. This figure, however, affords no reliable index, as is apparent from the wide variation in values corresponding to the different kinds of soil tested. The interpretation of the results as applying to design, therefore, is essentially a matter of judgment. The variations referred to are so great that the only safe course was to assume that the worst soil conditions (as regards frictional resistance) might exist under any bay. East of Buttress 36 (Plate IX) the clayey content of the overburden was somewhat greater than west thereof, and hence it seemed reasonable to assume that there the soil is less subject to percolation and has a somewhat greater bearing value. Nevertheless, no modification was made in the matter of frictional resistance on this account.

The coefficient of frictional resistance for the clayey foundation soil (wet) was, for design purposes, finally assumed to be 0.33. The writer believes that for none of the soils at the dam site is the corresponding coefficient less than 0.25.

Frictional Resistance of Shale.—Here, again, there are but few data available; perhaps the most pertinent are those given by Mr. E. L. Lasier in his article on "Tests of Frictional Resistance of Concrete on Shale."* These tests were made under the direction of the writer at the site of the State Line Dam of the Hydro-Electric Company of West Virginia, on Cheat River, in West Virginia, and cover, not merely the frictional resistance of concrete on shale, but also that of shale on shale.

* *Engineering News*, July 22d, 1915.

At the Stony River dam site the shales have pronounced horizontal laminations.* Hence the results in Mr. Lasier's Table 1 apply, the tests summarized in that table having been made on quarried blocks of shale. There the coefficient of frictional resistance for the shale specimens moving on the stationary shale blocks averaged 0.61 when the surfaces in contact were dry, and 0.55 when they were wet. In view of the apparent differences in physical characteristics, it seemed proper to adopt assumptions even more conservative than those shown in Mr. Lasier's tests. This is especially true as regards those Stony River shales near the surface of bed-rock. Inasmuch as these had been exposed to disintegration ages ago, clay seams or laminations had been formed. The coefficients adopted are given in Table 1.

Stability of Original Structure Against Sliding.—The stability of a dam against failure of a given kind is ordinarily measured by its so-called factor of safety—which perhaps may be more correctly termed its "ratio of safety". This is the ratio of the forces resisting failure in a certain manner to the forces tending to cause such failure; but, whether or not a ratio of safety really means anything depends on the assumptions made in deducing that ratio; and, as previously suggested, it is seldom that two engineers working entirely independently arrive at exactly the same results as regards the ratio of safety. The writer makes it a practice to study the stability of a structure under several different sets of assumed conditions. Sometimes, too, after having ascertained the ratio of safety under the conditions to which it appears a dam will be subjected, it is instructive to see how severe, perhaps even how unreasonably severe, the conditions can be made without reducing the ratio of safety below unity.

In the present instance the stability of the dam was examined under the assumption of a "normal maximum load" and also under the "most severe conditions within the limits of reason", all, of course, as they appear to the writer. These several load assumptions are set out in Table 3, for a typical section of the dam, *viz.*, at Bay 35 (Plate XII), and for the sections of maximum height, both on the basis of the original structure and also on the basis of the structure as strengthened during the reconstruction.

* Unlike the foundation shale at the Stony River site, the shales of the State Line site are not continuously laminated within themselves in any one plane. Hence the condition applying to the State Line site is more nearly that summarized in Table 2 of Mr. Lasier's article, which shows ratios of horizontal moving force to resistance to sliding (including resistance to shear in shale and concrete), the lowest of which is approximately 3.0.

TABLE 3.—Load Assumptions for Analyses of Stability of Stony River Dam.

Factors.	Original Structure.				Strengthened Structure.			
	Normal maximum load.		Most severe conditions within limits of reason.		Normal maximum load.		Most severe conditions within limits of reason.	
	Sections of maximum height.	Typical section at Bay 35.	Sections of maximum height.	Typical section at Bay 35.	Sections of maximum height.	Typical section at Bay 35.	Sections of maximum height.	Typical section at Bay 35.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
(*a*) Elevation of head-water	140.5	140.5	136.0	136.0	140.5	140.5	139.0	139.0
(*b*) Ice pressure, in pounds per linear foot	0	0	20 000	20 000	0	0	20 000	20 000
(*c*) Elevation of lower limit of equivalent full hydrostatic pressure against dam	89.5	99.0	85.0	95.0	91.0	99.0	85.0	95.0
(*d*) (1) Uplift pressure negligible.	Yes.	Yes.	No.	No.	Yes.	Yes.	No.	No
(2) Head, in feet, of equivalent uplift pressure (acting on one-half of affected area of base) at upstream edge of structure, decreasing uniformly to zero at downstream edge of footings, in case of original structure, and at deep drains in case of strengthened structure	.....	.....	48.0	39.0	.....	.....	54.0	44.0 *in re* sliding. 42.0 *in re* overturning.
(3) Length of base, in feet, over which uplift pressure is exerted	.....	.....	61.0	51.5	.....	.....	40.0	30.0
(*e*) Coefficient of frictional resistance	0.33*	0.33*	0.25*	0.25*	0.33	0.33	0.25	0.25

* Resistance (to horizontal movement) on part of the original unreinforced cut-off wall and the original toe probably inconsiderable and treated as forming part of the frictional resistance of the foundation soil at the base of the footings.

Note.—No tail-water pressure in any case.

Referring to Table 3, it should be noted that, in connection with the question of frictional resistance, no attempt has been made in the case of the original structure to determine the additional resistance of the original cut-off wall and toe. Inasmuch as the cut-off wall was unreinforced throughout, and was not tied into the body of the dam except at a few places, it is believed that the resistance of the cut-off

wall to horizontal movement was not a reliable factor, and that, even when present, it was inconsiderable in amount. Likewise, the resistance of the original toe was probably negligible, because the toe was relatively shallow.

The assumed conditions of Table 3, in so far as pressure is concerned, are entirely arbitrary. In view of the fact that, both at the sections of maximum height and at the typical section of Plate XII, the strengthened structure has an adequate cut-off, it is certain, of course, that the uplift pressure actually does not decrease uniformly from heel to toe, as has been assumed. However, everything considered, and especially in the absence of definite knowledge regarding the presence and character of the uplift pressure, it appears reasonable to base stability analyses on assumed "equivalent" conditions of uplift pressure (see Table 3, factor (*d*)-2). Manifestly, uplift pressure could not be exerted over the entire area of the base without causing flotation, which is impossible.

I.—Under "Normal Maximum Load".—Applying now the assumed "normal maximum load" conditions of Column (2), Table 3, to the maximum height sections of the original structure, and stating all quantities for one bay 15 ft. wide, the results are as follows:

Weight of concrete	1 044 900 lb.
Weight of water on deck	1 194 500 lb.
Sum of loads	2 239 400 lb.

Total resistance to sliding, $2\,239\,400 \text{ lb.} \times 0.33 = 746\,500$ lb., approximately.

$$\frac{\text{Forces resisting sliding}}{\text{Forces tending to cause sliding}} = \frac{746\,500 \text{ lb.}}{1\,218\,200 \text{ lb.}} = 0.61.$$

Or, *vice versa,* in order that this ratio may become equal to unity, the coefficient of frictional resistance must equal

$$\frac{0.33}{0.61} = 0.54.$$

II.—Under "Most Severe Conditions Within Limits of Reason".—Referring to the assumed conditions of Column (4), Table 3, the spillway crest (Elevation 136) has been adopted as the highest elevation at which ice pressure could be present, because flash-boards could

hardly have been used with safety on the original spillway. Uplift pressure has been assumed as acting along the entire width of the base of the original structure (though over only 50% of the area), because in severely cold weather the weep-holes through the footings were frozen up solidly and hence were ineffective. With all quantities stated, as before, in terms of a 15-ft. bay, the results are as follows:

Weight of concrete	1 044 900 lb.
Water load on deck	985 500 lb.
Total downward pressure	2 030 400 lb.
Deduct uplift pressure	675 000 lb.
Net load	1 355 400 lb.
Total resistance to sliding = 1 355 400 lb. $\times$ 0.25 =	338 850 lb.
Horizontal water pressure	1 219 200 lb.
Ice pressure	300 000 lb.
Total forces tending to cause sliding	1 519 200 lb.

Hence the ratio of safety is

$$\frac{338\,850 \text{ lb.}}{1\,519\,200 \text{ lb.}} = 0.223.$$

Or, in order that the ratio of safety against sliding may become equal to unity, the coefficient of frictional resistance must equal 0.25 divided by 0.223, *viz.*, 1.12. Evidently, therefore, the Stony River Dam as originally constructed could not be considered as having a reasonable margin of safety against sliding.

It would be interesting, of course, to know what coefficient of gross resistance to sliding obtained prior to failure. In order to derive the minimum value of such coefficient, the following assumptions are made, conforming to the conditions which actually existed, *viz.*:

(*a*) Head-water level at elevation of spillway crest, *viz.*, at Elevation 136;

(*b*) No ice pressure;

(*c*) Horizontal water pressure equivalent to full hydrostatic pressure of 46.5 ft., *viz.*, down to Elevation 89.5;

(*d*) Uplift pressure negligible;

(*e*) No tail-water pressure;

(*f*) Any resistance to horizontal movement furnished by the unreinforced cut-off wall or the toe-wall is treated as forming part of the gross resistance to sliding on the part of the foundation soil.

Under these conditions, the ratio of the force tending to cause sliding to the load above the base of the footings, for a 15-ft. bay, was

$$\frac{1\,013\,600 \text{ lb.}}{2\,030\,400 \text{ lb.}} = 0.50.$$

In other words, the coefficient of gross resistance to sliding, under the foregoing conditions, was at least 0.50; for, necessarily, the ratio of safety was at least unity.

Probably the resistance to horizontal movement, as expressed by such ratio or coefficient, was due in part to cohesion or resistance to direct shear within the foundation soil itself. That is, it is likely that failure would occur, not at the relatively rough surface of contact between the footing concrete and the foundation soil, but along an approximately horizontal plane in the body of the soil just below the base of the footings. Such resistance to shear will be discussed further in connection with the expedient adopted to increase the resistance to sliding in that portion of the dam which was not disturbed by the failure.

It should be noted that in the 65 days prior to the failure, during which period the dam withstood the pressure of the full reservoir, relatively cold weather obtained, and the ground-water was practically at its lowest level. In other words, it is likely that the frictional resistance would have decreased as the foundation soil became more saturated by leakage and ground-water during the following spring. Whether the dam would have stood, or how near it was to failure by sliding, no man can tell.

Means to Increase Resistance to Sliding.—Various expedients were considered for increasing the margin of safety of the original structure against failure by sliding. The method finally selected involved the construction of anchoring walls at the heel and toe of the original structure, essentially as shown on Plate XII, which shows typical sections of the dam, though not those of maximum height. In order to explain more clearly the reasons for adopting this method

of increasing resistance to sliding, the alternatives considered will be briefly described and their advantages and disadvantages stated.

1.—Referring first to the principal feature of the adopted means, the essential advantages of the anchoring wall at the heel (the action and construction of which are discussed in more detail later herein) are as follows (Plate XII):

(*a*) It utilizes the foundation soil under the dam, and in effect also a considerable quantity of the foundation soil immediately down stream from the dam, to increase the total weight which must be moved down stream in case the dam is to fail by sliding.

(*b*) The construction of such an anchoring wall at the heel seals the construction joint at the top of the original cut-off wall, this joint having in some places opened up sufficiently to allow appreciable, if not indeed serious, leakage.

(*c*) With such an anchoring wall, any down-stream movement on the part of the dam will tend to compact the underlying foundation soil, and, therefore, to make it less pervious.

(*d*) This was found to be the most economical method of obtaining the desired margin of safety against failure by sliding.

The only disadvantage of constructing an anchoring wall at the heel alone lay in the resulting high unit pressure against the clay immediately down stream from such "heel"—as the new anchoring wall at the heel of the dam has for convenience been styled.

2.—With especial reference to that portion of the original structure which remained intact, there was, of course, no opportunity to flatten the slope of the deck, and thus obtain a greater water load.

3.—The loading of the footings of the hollow structure, by using concrete, rock, or earth, was considered, and was rejected because:

(*a*) Such additional material would increase the load on the already too heavily loaded foundation soil;

(*b*) The original footings would have required even more strengthening than was considered necessary with the means finally adopted;

(*c*) Such loading would have required special provision for inspection of the drainage system; and

(*d*) The expedient was limited in extent, and, taken by itself, was not considered to be sufficient.

4.—Various structural additions at the toe of the dam were considered; among them an L-shaped anchoring wall such as that shown on Plate XII. This method of obtaining additional resistance to sliding was not open to the objections to the proposed loading of the original footings. However, if adopted to furnish the entire required increase in resistance to sliding, it would have been subject to certain different objections, *viz.*:

(*a*) For the same depth below original ground surface, generally speaking, less increase in resistance to sliding results than in the case of an anchoring wall at the heel alone; for the toe-wall does not utilize the foundation material under the body of the dam.

(*b*) In the case of this particular clayey soil, the toe-wall, like the "heel", is limited in efficacy by the safe bearing value of the soil in compression.

(*c*) In case of any yielding at the toe of the dam, either downward or down stream, the effect might be to cause either cracking in the original cut-off wall or, at the very least, the opening of the construction joint at the top of the old cut-off (Plate XII). Thus, water under reservoir pressure might immediately be admitted under the base of the dam.

On the other hand, the anchoring wall at the toe has certain advantages not possessed by the "heel", *viz.*:

(*d*) By means of its vertical member, the toe-wall confines the foundation soil under the original footings of the dam, thus increasing the safe load to which that soil may be subjected.

(*e*) The horizontal member of the toe-wall in effect increases the width of the base of the dam, thus decreasing the unit vertical load on the foundation soil, as will be shown later in more detail under the heading "Footings".

It is clear, however, that in cases where it is necessary similarly to increase the margin of safety of a dam against failure by sliding, but where there is no opportunity to construct an anchoring wall at the heel, an anchoring wall at the toe can be made to answer the purpose. Generally speaking, it is merely a matter of proportioning such a toe-wall properly.

5.—Investigation was made of the desirability of constructing reinforced concrete columns at the toe of the dam, inclining downward and approximately parallel to the line of action of the resultant load on the dam. This proposal was rejected, partly because the result would be equivalent to placing the dam on relatively unyielding supports—stilts, as it were—thus changing the character of the stresses in the original footings.

6.—The construction of a heavy block of concrete, approximately square in cross-section and extending along the entire length of the toe of the dam, was considered and rejected because the L-shaped toe-wall transmits vertical and horizontal loads into the foundation material to the same extent, but with much less concrete and excavation.

In view of the foregoing considerations, it was concluded to use anchoring walls at both heel and toe; for, by means of the toe-wall, it was possible to reduce to a safe value the otherwise excessive horizontal load on the foundation soil at the heel. The reconstructed dam acts essentially as a monolith; hence down-stream deflection or yielding must be equal at heel and toe. Inasmuch, then, as the load on the foundation soil is proportional to the deflection, the unit horizontal loads at heel and toe are approximately equal.

Likewise, at the new spillway of the dam, an anchoring wall at the heel was used. Here advantage was taken of the opportunity of combining the cut-off and the anchoring device in a single wall, as shown on Plate X. As the foundation of the new spillway consists of shale, the problem of keeping within the safe load against the material at the heel was not difficult, and hence no anchoring wall was required at the toe.

Analysis of Anchoring Walls.—The principle on which the design of the anchoring walls is based applies both in the case of the strengthening of the original structure and in that of the construction of the new spillway. The detailed explanation, however, will be made with reference to the typical bulkhead section at Bay 35, as shown on Plate XII.

The use of such anchoring walls, to furnish specific and reliable resistance to sliding, by utilizing the weight of the foundation material underlying, and immediately down stream from, a dam, is believed to be new. The writer first applied it in the case of the State Line Dam of the Hydro-Electric Company of West Virginia, on Cheat River in West Virginia.

Referring now to Plate XII, the anchoring wall at the heel is tied into the dam by twisted, square, steel bars extending through openings broken through the base of the original deck of the dam. The heel itself consists essentially of two cantilever members of reinforced concrete, extending, respectively, above and below the steel which ties the heel to the original structure. A relatively long cantilever member extends downward immediately adjacent to the up-stream face of the original cut-off, and a relatively short cantilever member extends upward immediately adjacent to the base of the original deck. Both cantilevers are suitably reinforced against stresses due to bending moment and shear.

Although some of the horizontal water pressure is exerted directly against the anchoring wall or heel, the greater portion is exerted against the deck, and by the deck is transmitted into the buttresses. The tie-steel previously referred to is embedded in the reinforced concrete footing strengthening, which in turn is thoroughly tied into the base of each adjacent buttress, as shown on Plate XII. Thus the tie-steel receives from the buttresses and the footings that portion of the horizontal water pressure which is not exerted directly against the heel, or has not been taken up by the frictional resistance of the foundation soil immediately in contact with the footings.

The lower cantilever member of the anchoring wall in turn transmits such surplus load into the foundation soil down stream from the heel. The fact that the stress is transmitted through the medium of the original cut-off wall is immaterial, except to the extent that the cut-off wall serves to transmit some of the load into the foundation soil below, and down stream from, the bottom of the heel. The reaction at the top of the lower cantilever, of course, is taken up by the upper and shorter cantilever and transmitted through the deck, in part directly into the buttresses and in part into the approximately triangular fillet of new concrete placed in the angle between the deck and the footings (Plate XII). Ordinarily, this fillet acts as an arch to transmit the reaction back into the buttresses and footings. In certain cases, however, where steps occur in the footings at the centers of bays, it was necessary to reinforce the triangular fillets to act as cantilever beams supported at the buttresses and extending outward to the centers of the adjacent bays.

The effect of the anchoring wall at the heel is essentially to lower the main "plane of least resistance" to sliding from its original posi-

tion, at or immediately under the base of the footings of the dam, to a position approximately at the elevation of the bottom of the new anchoring wall at the heel. By "plane of least resistance" to sliding is meant the surface or surfaces, approximating a plane or planes extending through the foundation material, along which the total resistance to horizontal movement (sliding) is less than along any other surfaces or approximate planes.

In so far as resistance to sliding is concerned, the result of such lowering of the main "plane of least resistance" is to cause the weight of the foundation material in advance of the anchoring wall to be utilized at least as effectively as the weight of the body of the dam itself. This utilization of the weight of the foundation material occurs in two ways, as will be apparent by reference to Plate XII. Let it be assumed that, for the conditions existing in the typical section there illustrated, the "planes of least resistance" are represented by the lines, *A-B* and *B-E*, respectively. Then

1.—The foundation material lying above *A-B* and to the left of *B-C* must, in case of failure, slide along the plane, *A-B*. This is also true of the concrete structure (including the anchoring wall) and the superimposed water load. The resistance to sliding of these several elements is represented by the product of their weight multiplied by the coefficient of frictional resistance.

2.—The foundation material lying between *B-C* and *B-E* must, in case of the failure of the dam by sliding, move simultaneously down stream and upward along the plane, *B-E*. Such movement of this material, therefore, is opposed, not merely by frictional resistance, but also by the force of gravity. The resistance of this particular portion of the foundation material is analyzed subsequently in more detail.

The location of the "planes of least resistance" is dependent on the conditions existing in any particular case. For the conditions shown on Plate XII, a general statement may be made, as follows:

It is evident that, assuming proper design and construction of the anchoring wall at the heel of the dam, the main "plane of least resistance" cannot begin above *A*. Neglecting for the moment the existence of the original cut-off, it is further evident that the main "plane of

least resistance" could not extend downward in a direction such as *A-G*. In other words, the resistance along any plane dipping below the horizontal is necessarily greater than the resistance along the plane, *A-B*. Whether the "plane of least resistance" extends horizontally or in an upward direction from *A* is a matter for determination by trial computation. In this particular case, of course, the old cut-off wall is a factor, even though its effect may be indefinite. For instance, were the break in it to be in an upward direction, such as *A-F*, its effect on the location of the main "plane of least resistance" would be nil, or at least negligible. On the other hand, a break in a downward direction, such as *A-H*, might, owing to the strength of the concrete cantilevering below the elevation of *A*, cause the initial point of the main "plane of least resistance" to be at a lower elevation, for example, at *H* instead of *A*.

To the left of the vertical plane represented by the line, *B-C*, passing through the down-stream edge of the new toe-wall of the dam, the foundation material is confined by the weight of the structure shifted, as it were, toward the down-stream side of the footings by the horizontal thrust of the water load. In effect, this weight is greatest per square foot at the very down-stream edge of the toe-wall, which is practically monolithic with the original structure. The line or plane of failure could not turn upward (from the main "plane of least resistance") on the left side of *B-C* unless the foundation material underlying the dam, and above such new and increased slope, were to fail in compression, or were to "flow".

Moreover, at the toe there is a concentration or transfer of horizontal thrust into the foundation material. In the present instance this transfer of thrust is accomplished partly through frictional resistance between the soil and the toe-wall (especially the horizontal leg thereof), and partly through direct bearing of the toe-wall (especially the vertical leg thereof) against the soil.

It is probable, therefore, that the main "plane of resistance" would end approximately below the down-stream edge of the toe-wall, as at *B*.

If, then, the main "plane of least resistance" is represented by *A-B*, it follows that unless (1) the foundation material fails in compression, as along *B-C*, or (2) the coefficient of frictional resistance is too great, the "plane of least resistance" will turn upward at *B*. Generally speaking, the slope of this plane must be determined by trial,

although, under certain conditions, the slope may be determined by direct computation.

Theory of Resistance at Toe.—The resistance of the foundation material at the toe of the dam, by which, on Plate XII, for instance, is meant the material to the right of *B-C*, is a factor sufficiently important to warrant more detailed explanation. Practically, on account of the character of the material in question, the results of theoretical analysis represent the actual conditions only approximately. The refinement of such analysis, however, is worth while, because it aids in obtaining a thorough understanding of the case.

The problem is essentially one of "passive thrust". The analysis adopted as applying to the general case is based on the assumptions that:

(*a*) The foundation material, *viz.*, earth, is homogeneous and granular;

(*b*) The material is without cohesion;

(*c*) The coefficient of frictional resistance (being the same as the coefficient of internal friction) $= f$;

(*d*) In case of sliding, failure of the foundation material at the toe will occur along a plane, such as *B-E*;

(*e*) The resistance of the foundation material to compression (as in the vertical plane, *B-C*) is so great as not to be the limiting factor in toe resistance;

(*f*) The load on the foundation material in a vertical plane (such as *B-C*) increases in intensity regularly from zero at *C* to a maximum at *B*;

(*g*) It is necessary to the stability of a granular mass that the direction of the pressure of the portions into which it is divided by any plane should not at any point make with the normal to that plane an angle exceeding the angle of repose, *viz.*, *f*.*

Under the foregoing assumptions, it would appear that, in case of failure of the dam by sliding, a wedge-like section of the foundation material would be forced upward approximately between and along *B-C* and *B-E*. The toe resistance—in other words the resistance to movement on the part of such wedge-like section—depends on whether there is frictional resistance along the vertical plane, *B-C*, such as

* Rankine's "A Manual of Applied Mechanics", 17th Ed., p. 213, Theorem I.

there is along the inclined plane, *B-E*. There appears to be no valid objection to the assumption that the resistance along the vertical plane is equal to that along the inclined plane. However, the results will be stated on each basis, the more conservative assumption being that there is no frictional resistance in the vertical plane, *B-C*:

Condition I.—Without friction in the vertical plane, *B-C*. (See Fig. 20.)

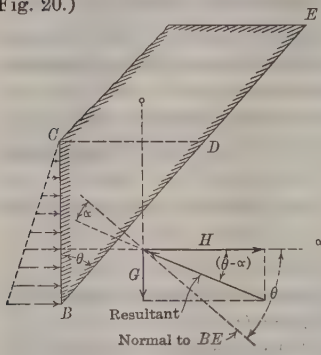


FIG. 20.

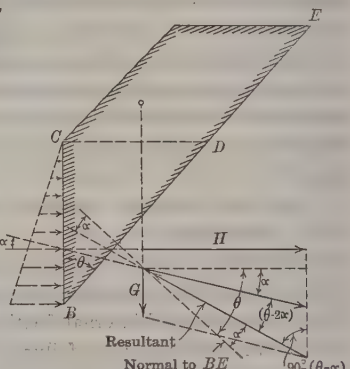


FIG. 21.

If H = horizontal pressure in the plane, *B-C*, against the wedge of earth, *C-B-E*, sufficient to cause movement of the wedge;

G = weight of material (per unit length of dam) included between *B-C* and *B-E*;

α = angle of friction (*viz.*, the angle the tangent of which is f);

θ = angle between "plane of least resistance", *B-E*, and the vertical (plane *B-C*).

Then $H = G \cot. (\theta - \alpha)$, since the friction along the plane, *B-E*, resists both the thrust, H, and movement of the weight, G.

For the purpose in question, it is necessary to determine the minimum value of H. In the general case, with an irregular surface boundary between *C* and *E*, it is necessary to determine such a minimum value by trial. Ordinarily, this can most easily be accomplished by plotting a curve with values of θ as abscissas, and corresponding computed values of H as ordinates. The lowest value of H shown by such a curve is, of course, the desired value.

However, in the special case in which the upper bounding surface is horizontal—as represented on Plate XII and Fig. 20 by *C-D*—the minimum value of H may be stated mathematically by expressing G in terms of θ and then placing the first differential of the resulting equation equal to zero. Accordingly, it is found that H is a minimum when

$$\theta = 45° + \frac{\alpha}{2}.$$

Evidently, in this special case the critical value of θ (giving the minimum value of H) cannot be less than 45 degrees.

Condition II.—The frictional resistance in the vertical plane, *B-C*, equals f. (See Fig. 21.)

In the general case, under this condition,

$$H = G \cos. \alpha \frac{\cos. (\theta - \alpha)}{\sin. (\theta - 2\alpha)}$$

It follows that, since H becomes infinitely great when $\sin. (\theta - 2\alpha) = 0$, θ must be greater than 2α. Again, under ordinary physical conditions, G and, hence also, H, become infinitely great for $\theta = 90$ degrees. Therefore, excepting for failure in compression, failure cannot occur unless α is less than 45°—in other words, unless f, the tangent of α, is less than 1.0.

In the special case where the upper surface of the toe material lies in the plane, *C-D*, the minimum value of H may be determined mathematically, as under Condition I, it being found that H is a minimum when

$$\theta = \tan.^{-1}\left(\frac{2 \tan. \alpha + \sec. \alpha \sqrt{2}}{1 - \tan.^2 \alpha}\right)$$

Actual Toe Resistance.—Under many, if indeed not under most, foundation conditions, the toe resistance is limited practically, not by the theoretical minimum values of H as expressed in the foregoing paragraphs, but by a third condition, namely, the bearing value of the foundation material. This is the case at the Stony River site. For design purposes, the maximum load in the plane, *B-C*, under the conditions illustrated on Plate XII, was limited to the safe bearing value, in lateral compression, of the soil at the site. This value was assumed as 4 000 lb. per sq. ft. In other words, the average resistance to sliding in the vertical plane, *B-C*, was limited to 2 000 lb. per sq. ft.

To illustrate the possibilities of toe resistance, as well as the probable actual values, reference is again made to the typical section of the strengthened structure at Bay 35, as shown on Plate XII, all calculations being based on normal maximum load conditions, with f assumed $= 0.33$:

Assuming first that the main "plane of least resistance" to sliding is horizontal (see *A-B*, Plate XII), and assuming that there is no frictional resistance in the vertical plane, *B-C*, the inclined "plane of least resistance" at the toe is found to make an angle, θ, of 44° with the vertical for the minimum value of H (toe resistance to sliding). The corresponding total resistance to sliding is approximately 140 080 lb. per lin. ft. If, instead, the frictional resistance in the vertical plane, *B-C*, is assumed to be equal to that in the inclined "plane of least resistance", the latter is found to make an angle of 67° with the vertical, with a corresponding total resistance of about 183 530 lb. per lin. ft. However, if limited by the assumed safe bearing value of the soil in lateral compression, that is 4 000 lb. per sq. ft., the total resistance to sliding is only 115 080 lb. per lin. ft.

Assuming now at random a different location of the main "plane of least resistance", *viz.*, *A-I* (Plate XII), making an angle, β, of 6° with the horizontal, it is found that, without frictional resistance in the vertical plane, *I-C*, the angle, θ, is 39°, and the total resistance to sliding is about 131 200 lb. per lin. ft.; that with full frictional resistance ($f = 0.33$) in the plane, *I-C*, the angle, θ, is 65°, and the corresponding total resistance is 159 300 lb. per lin. ft.; but that, if limited by the safe bearing value of the soil, the total resistance is only 121 650 lb. per lin. ft. In order to determine the minimum values of the total resistance to sliding for each of the three conditions just mentioned, enough different locations of the main "plane of least resistance" were assumed, and corresponding calculations were made, to warrant the plotting of a diagram in which the values of β (the angle which the main "plane of least resistance" makes with the horizontal) and of total resistance to sliding, respectively, were plotted as rectangular co-ordinates.

It was evident from inspection of the results that, with the resistance at the toe limited by the safe bearing value of the soil, the minimum total resistance to sliding is afforded when the main "plane of least resistance" is in the horizontal position, *A-B*. However, from

the diagram which was based on the assumption that there is no frictional resistance in the vertical plane, β was found to be approximately 8° for the minimum value of total resistance to sliding, equaling about 128 500 lb. per lin. ft. Correspondingly, a value of 8° 15′ was found for β under the assumption that the coefficient of frictional resistance in the vertical plane too is 0.33, the minimum total resistance to sliding being approximately 152 700 lb. per lin. ft. Both these values of total resistance to sliding are greater than that obtained when the resistance at the toe is limited by the safe bearing value of the soil, *viz.*, 115 080 lb. per lin. ft. Consequently, for design purposes, the "planes of least resistance" under "normal maximum load" for the section shown on Plate XII were assumed to be represented by *A-B* and *B-C*.

It is interesting to note to what extent the total resistance to sliding at Bay 35 was increased in the reconstruction. Retaining still the "normal maximum load" conditions, and with resistance at the toe limited by the safe bearing value of the soil, the total resistance to sliding was increased from approximately 41 310 lb. per lin. ft., in the case of the original structure, to approximately 115 080 lb. per lin. ft., in the case of the strengthened structure. In other words, the total resistance to sliding was increased by about 178 per cent. Correspondingly, on the basis of the assumption that the resistance at the toe is not limited by the safe bearing value of the soil, but that no frictional resistance exists in the vertical plane at the toe, the total resistance to sliding would be increased by about 211%; and if, in addition, the coefficient of frictional resistance in the vertical plane is assumed to be 0.33, the increase would have been about 270 per cent.

Of course, there is undoubtedly a considerable difference between theoretical and actual conditions. Thus, referring to the foregoing assumptions on which the theoretical analysis has been based, earth is rarely, if ever, strictly homogeneous. The coefficient of frictional resistance varies, and the "planes of least resistance" are probably never true planes. Crude experiments made by the writer indicate that, even in the case of such granular material as sand, the surface along which failure occurs is by no means a true plane. Under the Stony River conditions, the "planes of least resistance" are affected also by boulders. Again, any cohesion existing within the founda-

tion soil causes the resistance to sliding to be greater than according to the preceding analysis. It is evident, however, that procedure along the lines indicated is amply conservative and safe.

It may not be amiss to point out that anchoring walls such as those described may be applied properly and economically in the case of rock foundations, particularly when the foundations consist of laminated rock with relatively low frictional resistance. By this means a considerable quantity of the foundation rock can be made to afford resistance to sliding just as reliably as though the corresponding weight consisted of concrete in the body of the dam. Generally speaking, anchoring walls, even if not used for the purpose of reducing the quantity of material in the body or superstructure of a dam, will, at least, increase the margin of safety.

Stability of Strengthened Structure Against Sliding.—The design of the features entering into the strengthening of those portions of the dam which remained intact conformed as closely as practicable to the general criterion that the ratio of the probable forces resisting sliding should be twice the probable forces tending to cause sliding.

It will be noted from Plate XII that an earth fill has been placed down stream from the typical section there shown. Fig. 22 shows more clearly the limits and height of this fill, which extends along the entire length of the higher sections of the dam, *viz.*, between the old and the new spillways. Referring again to Plate XII, it is evident that the presence of this earth fill increases the quantity of material lying within the angle, θ. Moreover, the greater the coefficient of frictional resistance of the foundation soil, the greater the angle, θ, and hence the greater the effective resisting weight due to the earth fill. The material for the fill was obtained largely from the excavation for the new spillway channel.

As in the case of the original structure, the margin of safety resulting from the work of strengthening will be illustrated by several examples under different sets of conditions. Thus, for the strengthened sections of maximum height, the results are as follows:

I.—Under the assumed "normal maximum load" conditions, as set out in Column (6), Table 3,

$$\frac{\text{Forces resisting sliding}}{\text{Forces tending to cause sliding}} = 2.03.$$

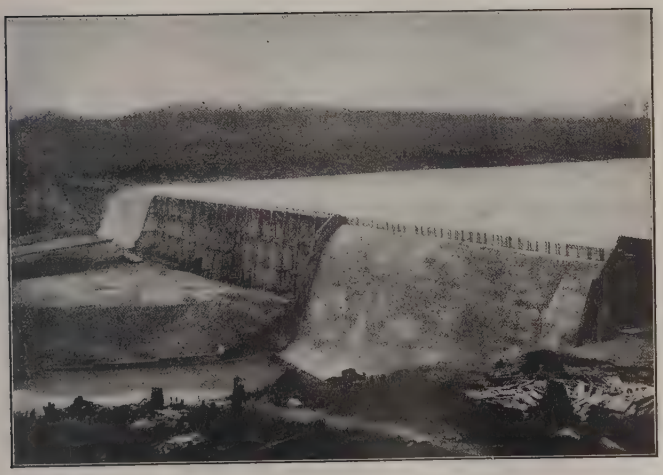
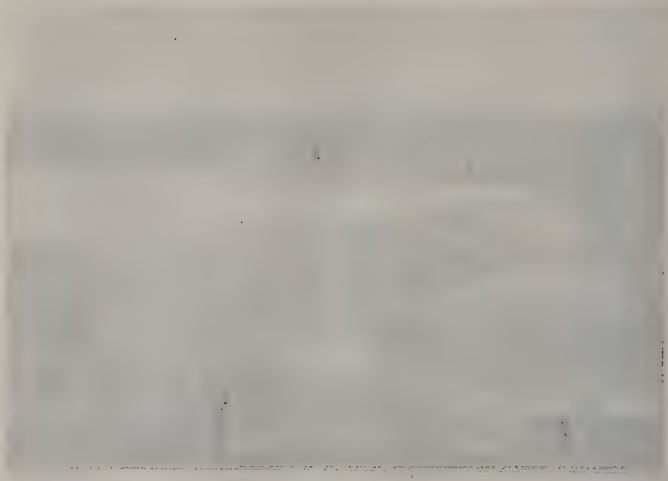


FIG. 22.—STONY RIVER DAM AS RECONSTRUCTED. VIEW FROM WEST BANK.

FIG. 23.—EROSION UNDER FOOTING AT HEEL OF DAM.



For the original—that is, the unstrengthened—structure, the corresponding ratio, as previously explained, was 0.61. By comparison of the conditions set out in Columns (2) and (6), Table 3, it will be noted that the only difference in the assumed conditions is that in the case of the strengthened structure, Column (6), factor (*c*), the assumed elevation of the lower limit of equivalent full hydrostatic pressure against the dam was raised somewhat. This was warranted by the fact that considerable pervious material was taken out of the river-bed, immediately up stream from the dam, and replaced by a back-fill of impervious clay, the back-fill in turn being covered in part with a reinforced concrete protecting mat, especially near the sluice-gates (see Plate XI).

II.—Under the "most severe conditions within the limits of reason", as applied to the strengthened structure (see Column (8), Table 3),

$$\frac{\text{Forces resisting sliding}}{\text{Forces tending to cause sliding}} = 1.07.$$

This ratio is to be compared with the ratio of 0.22 for the sections of maximum height, of the structure as it stood originally. The comparison is the more striking when it is noted that, inasmuch as 3-ft. flash-boards can be used safely on both spillways of the reconstructed dam, the assumed elevation of head-water is 3 ft. greater in the case of Column (8) than in the case of Column (4), which correspondingly sets out the "most severe conditions within the limits of reason" for the sections of maximum height of the original structure.

As to the typical strengthened section at Bay 35, shown on Plate XII, the margin of safety against sliding is represented as follows:

III.—Under the "normal maximum load" conditions set out in Column (7), Table 3, the ratio of safety is 2.14.

IV.—Under the "most severe conditions within the limits of reason", as set out in Column (9), Table 3, the ratio of safety is 1.12.

These ratios, of course, would be bettered by any increase in the respective coefficients of frictional resistance assumed. There are, however, certain additional factors, tending toward greater safety against sliding, which apply in the case of the strengthened portions

of the dam founded on the clayey over-burden, but in none of which was any specific reliance placed, namely:

(*a*) In the structure as strengthened there is no tendency to break at the top of the cut-off wall. The resistance of the old cut-off wall to shear or rupture at the bottom of the new "heel" is, therefore, a considerable factor in resisting sliding.

(*b*) Again, if such shear or rupture in the cut-off wall should occur below the bottom of the "heel", or should extend below the bottom of the "heel" in a direction such as *A-H*, Plate XII, the effect would be to lower still further the "planes of least resistance" and, consequently, to increase the resistance to sliding.

(*c*) The existence of cohesion or resistance to shear in the clayey foundation soil is by no means speculative even though the amount of such resistance may not be readily and reliably determinable.

Tests of Shearing Value of Clay.—The writer has found no previously published data on this subject, and therefore believes that certain tests made during the Stony River reconstruction by Mr. D. N. Showalter, Resident Engineer, will be of general interest. The apparatus contrived by Mr. Showalter for making the test was simple and yet quite effective.

Specimens of clay were dug out of the foundation over-burden and cut to such shape as to fit the apparatus. The shearing was then accomplished by slicing off the clay, as it were, with a 2 by 6-in. piece of wood which was forced across the specimen by a small set of blocks and tackle. The tension in the rope was measured by a spring balance. The specimens tested were fairly moist. The results of these tests of the shearing values of several different clayey soils are given in Table 4.

From inspection of the data in Table 4, it will be noted that, as was to be expected, the presence of sand in the clay very materially lessens the shearing value. However, even among Specimens 1 to 8, inclusive, there is considerable variation among the results. The effect of the length of time of application of a given shearing load was not investigated, though time is undoubtedly a very important

factor. The results clearly indicate that here is a fruitful field for further experimentation.

TABLE 4.—Results of Tests of Shearing Value of Foundation Soils at Stony River Dam Site.

Experiment No.	Character of soil under test.	Initial area under shearing stress, in square inches.	Load* at failure, in pounds.	Shearing value, in pounds per square inch of initial areas.
1	White clay.	9.0	109	12.1
2	" "	4.8	128	26.6
3	" "	6.1	126	20.7
4	" "	8.7	226	26.0
5	" "	7.7	176	22.8
6	Black gumbo.	19.2	184	9.6
7	" "	19.2	309	16.1
8	Black loam.	13.7	219	16.0
9	Sandy yellow clay.	19.2	89	4.6
10	" " "	14.6	89	2.7
			Average.....	15.7

* Including proper allowance for weight of apparatus.

Note.—All soil was in natural state of moistness.

To the writer, it appears that it is hardly proper, in the light of present information, that the shearing value of clay should be taken into account over and above the resistance to sliding developed under such conditions as obtained in the tests of frictional resistance reported in Table 2. The resistance of the clay to shear, however, affords an added margin of safety, indefinite though the extent of the added margin may be. Accordingly, it is interesting to consider what might be the effect of such resistance to shear in a typical case, for example, the section at Bay 35 under "normal maximum load" conditions.

Referring to Plate XII, it will be remembered that, in case of failure of the dam by sliding, it was assumed that failure along *A-B* would occur by sliding of clay on clay, and that failure along *B-C* would occur by the crushing of the clay or other foundation material. Hence it is only along *A-B* that the resistance of the clay to shear could increase the total resistance to sliding. Such resistance to shear, however, would act instead of, and not in addition to, the frictional resistance along *A-B*, which, under the assumed conditions, is about 75 000 lb. per lin. ft. of dam. The average shearing value shown by the tests reported in Table 4 is 15.7 lb. per sq. in. If, then, a resistance to shear of 15 lb. per sq. in., for instance, existed in the

53.5 lin. ft. of clay along *A-B*, the total resistance along *A-B* (exclusive of resistance to rupture on the part of the old cut-off wall) would be about 115 000 lb. per lin. ft. of dam. Thus the total resistance to sliding would be increased by about 40 000 lb. per lin. ft.

However, for the reasons stated previously, and because of the varying characteristics of the foundation soil, resistance of the clayey soil to shear was not relied on to furnish resistance to sliding.

Resistance to Sliding at New Spillway.—Turning now to the new spillway section, as shown on Plate X, it will be noted that, by reason of the use of the combined anchoring wall and cut-off at the heel of the structure, the principal "plane of least resistance" (at the elevation of the bottom of the anchoring wall) lies in relatively hard shale. Free from the hampering conditions imposed by a structure already existing on the site, it was possible at this particular place to secure more readily and economically given ratios of safety than obtain in those portions of the dam which were left intact after the failure. Thus the new spillway section has generally a greater margin of safety.

I.—Referring to the 15-ft. wide section centering on Buttress 17 (Plate IX), the ratio of safety against sliding under the "normal maximum load conditions" is 3.69. The conditions assumed in arriving at this ratio were essentially like those of Column (6), Table 3, except that, in accordance with Table 1, the coefficient of frictional resistance was assumed to be 0.5.

II.—For the same location in the new spillway, but under the "most severe conditions within the limits of reason," the ratio of safety against sliding is 2.03. In this case the assumed conditions were essentially like those of Column (8), Table 3, except that

(*a*) Head-water was assumed at Elevation 142.25, with no ice pressure; and
(*b*) The minimum coefficient of frictional resistance, in accordance with Table 1, was assumed as 0.40.

It is pertinent at this point to note, by reference to Plate XII, that apparently no harm would result from uplift pressure exerted along the planes of contact between the footings of the strengthened structure and the foundation soil. That is, there would be practically no change in the vertical load above the probable "plane of least resist-

ance", such as *A-B*. Likewise, referring now to Plate X, it would appear that uplift pressure could exist in the laminations of the foundation shale rock immediately under the buttress footings (but above the elevation of the bottom of the anchoring wall) without affecting materially the stability of the new spillway section as regards sliding. Such uplift, however, would have a material effect on stability as regards overturning.

Resistance to Overturning.

In the case of a hollow dam with a deck slope of approximately 45° from the vertical, it is hardly necessary to inquire deeply into the question of stability as regards overturning. This will be apparent from the following illustrations:

I.—In the case of the typical section at Bay 35, in its original, unstrengthened condition (see unshaded portion of Section *B-B*, Plate XII), the ratio of safety against overturning under the assumed "normal maximum load" conditions of Column (3), Table 3, was 3.65.

II.—For the same section under the "most severe conditions within the limits of reason", as set out in Column (5), Table 3, the corresponding ratio of safety was 1.14.

In deriving these ratios, moments were taken about the point, *O*, at the toe of the original structure, as indicated in Section *B-B* of Plate XII, and it was assumed that, should the dam begin to overturn, the original cut-off wall would fail at the construction joint at the top of the wall. Furthermore, the horizontal thrust was considered to be taken up entirely by frictional resistance along the base of the footings; that is, no resistance to horizontal thrust was attributed to the body of the cut-off wall. In the second of the foregoing sets of assumptions cognizance was taken of the fact that, in severely cold weather, the weep-holes in the footings of the original structure would have frozen up solidly and become ineffective. Consequently, uplift pressure was considered as affecting a portion, assumed to be 50%, of the total area of the base.

Referring now to the typical strengthened structure at Bay 35, as shown on Plate XII:

III.—The ratio of safety against overturning under the assumed "normal maximum load" conditions of Column (7), Table 3, is 4.57.

IV.—Under the "most severe conditions within the limits of reason", Column (9), Table 3, the corresponding ratio becomes 1.68.

In developing the last two ratios it was assumed, as before, that in case of actual overturning, the original cut-off wall would fail at the construction joint at the top of the wall. However, as regards the effect of frictional resistance along the base of the main footings of the dam, in taking up horizontal thrust, such resistance was assumed to be equal to only 0.2 of the total concrete and water load, the remainder of the horizontal thrust causing a uniformly distributed load on the anchoring wall at the heel. The anchoring wall at the toe was assumed to carry no load. Such assumed distribution of the resistance to the horizontal thrust is obviously arbitrary, but affords the most unfavorable conditions as regards stability against overturning. The resultant of the assumed load on the "heel" would necessarily act at half the depth of the "heel".

Needless to say, the increased ratios of safety against overturning in the case of the strengthened structure were not sought after *per se*. They resulted from the provision made to increase the margin of safety against sliding.

Footings.

Bearing Value of Foundation Soil.—In the case of clayey soil, compressive loading causes appreciable yielding; hence the safe loading or bearing value is measured by the maximum allowable yielding. As the result of observations made at the dam site, under various conditions during the investigation period, and also of various tests of the soil at the site, it was concluded that the yielding due to a compressive loading of about 5 000 lb. per sq. ft. would not exceed 0.1 in. provided the soil is confined against "flow".

It was considered preferable not to exceed a maximum loading of 4 000 lb. per sq. ft., but somewhat greater loads were allowed for the horizontal bearing of the anchoring walls at heel and toe, where the soil is well confined, and also for the footings at the eastern portion of the dam, where the foundation material is of manifestly better character.

The results of the foregoing tests of the safe bearing value of the soils at the site are given in Table 5. The following comments apply to the table:

1.—The variations among the results are characteristic of the foundation soils in question.

2.—In view of the fact that the areas under load in the several tests were very small, it is reasonable to assume that under the actual conditions of loading the foundation soil would show less yielding for a given load. Generally speaking, the more confined the soil under compression, the less its yielding under load, and hence the higher its safe bearing value; furthermore, the greater the area under compression, the greater the proportion of the stressed soil which is confined against "flow".

3.—In general, the foundation soil of the east bank at the dam site has a higher bearing value than that of the middle of the valley, or of the west bank.

Loading of the Foundation Soil.—Calculations as to the loading of the foundation soil were based on the assumption that where the cut-off, whether in its original state or underpinned, is of relatively great depth, and especially where it extends down to bed-rock, the cut-off wall forms a relatively rigid, unyielding support for the dam; whereas the foundation soil down stream from the cut-off forms a yielding support. For practical purposes, therefore, it has been assumed that, when under load, the dam pivots about the top of the original concrete cut-off wall, the cut-off and the foundation soil down stream therefrom each carrying its due proportion of the total vertical load. The axis about which the dam is assumed to pivot is approximately in the plane of the base of the original footings. Following this assumption further, it is evident that the deflection in the foundation soil would vary approximately uniformly from zero at about the center of the cut-off wall to a maximum at the down-stream edge of the toe of the dam.

There is, of course, a certain amount of flexibility in the concrete footings; that is, the true deflection (and hence stress) diagram of the soil underlying the structure would not be a straight line. However, the effect of such flexibility in the footings is indeterminate, and the straight-line assumption was adopted in computations.

TABLE 5.—Results of Tests of Bearing Value of Foundation Soils at Stony River Dam Site.

Experiment No.	Character of soil under test.	Yielding of Soil, in Inches. Load, in Pounds per Square Foot.											
		1 000	2 000	3 000	4 000	5 000	6 000	7 000	8 000	9 000	10 000	11 000	12 000
1	Black loam	.....	.....	0.004	0.018	0.040	0.079	0.093	0.100	0.138	0.152	0.156	.....
2	Sandy yellow clay	0.002	0.018	0.045	0.086	0.093	0.141	0.168	0.177	0.179	0.186	0.206	0.230
3	Hard yellow clay containing small stones	.....	0.007	0.020	0.029	0.045	0.066	0.074	0.080	0.117	0.153	0.248	.....
4	Brown clay	.....	.....	0.003	0.009	0.060	0.133	0.218	0.300	0.372	0.498	.....	.....
5	Yellow clay	.....	.....	0.007	0.008	0.021	0.028	0.043	0.056	0.070	0.078	0.103	.....
6	Black gumbo	.....	0.027	0.058	0.102	0.159	0.231	0.313	0.407	0.500	.....	.....	.....
7	do.	.....	0.001	0.015	0.041	0.074	0.091	0.094	0.119	0 155	0.206	0.244	.....
8	Yellow clay, containing pebbles	0.006	0.031	0.046	0.090	0.117	0.170	0.184	.....	.....	.....	.....	.....
9	do.	.....	.....	.....	.....	.....	.....	0.153	0.158	0.162	0.167	0.173	0.181

Note.—The figures showing the yielding were taken from plotted results of the tests. The area of soil under load was approximately 3.6 sq. in. in all experiments, except that in No. 6 the area under load was 35.2 sq. in. In no case was the soil under load confined against lateral "flow."

The results stated below were derived on the basis of the further assumption that the original, as well as the strengthened, footings were "continuous;" in other words, that the footing for each buttress was 15 ft. wide. As a matter of fact, this assumption is hardly warranted, in so far as the original structure is concerned, as will be discussed in more detail later herein. Considering the loads to be carried by the buttress footings alone, the maximum soil pressures for the original structure would be approximately double those stated.

Referring to the typical sections of Plate XII, the computed pressures on the foundation soil, in the case of the original structure at Bay 35, are as follows:

I.—Under the "most severe conditions within the limits of reason", as set forth in Column (5), Table 3, with the exception that uplift pressure is considered to be negligible, the maximum vertical pressure on the foundation soil is 3 560 lb. per sq. ft.; the corresponding slant pressure is 6 000 lb. per sq. ft.

The foregoing assumptions, as also those in the three cases following, treat the horizontal thrust as taken up entirely by the frictional resistance of the foundation soil at the base of the footings. If the uplift pressure of Column (5), Table 3, were to be taken into account, the result would be to diminish the previously stated maximum soil pressures somewhat, but not to a considerable extent.

II.—If, for the sake of comparison, one similarly applies to Bay 35 of the original structure the more severe conditions which, according to Column (9), Table 3, are assumed to apply to the same section of the strengthened structure, the maximum vertical pressure on the foundation soil becomes 3 930 lb. per sq. ft.; and the corresponding slant pressure becomes 6 740 lb. per sq. ft.

The maximum soil pressures at the maximum height sections of the original structure are approximately the same as those indicated above for the typical section.

Due largely to the use of the new toe-wall (Plate XII), the unit compressive loads on the foundation soil were reduced considerably by the reconstruction, despite the fact that under the "most severe

conditions" the strengthened structure is considered to be subject to higher loading than the original structure. Thus:

III.—Under the "most severe conditions within reason", as set forth in Column (9), Table 3 (but disregarding any uplift pressure), the maximum vertical soil pressure for the strengthened structure at Bay 35 is 3 560 lb. per sq. ft.; the corresponding slant pressure is 4 795 lb. per sq. ft.

The latter pressure should be compared with that of 6 740 lb. per sq. ft. derived for the original structure under the same assumptions of loading (Case II). If, in Case III, uplift pressure had been assumed to be present, the results would not have been altered materially, because, in view of the assumed pivoting about the cut-off wall, uplift pressure under the "heel" would tend to increase the maximum compressive load on the foundation soil, whereas uplift pressure under the original footings down stream from the cut-off would tend to decrease such maximum load. Had the new "heel" been assumed to take up a portion of the horizontal thrust, the result would have been to increase the maximum vertical, but to decrease the maximum slant soil pressures.

IV.—Under the "normal maximum load" conditions of Column (7), Table 3, the maximum vertical soil pressure for the strengthened structure at Bay 35 is only 3 105 lb. per sq. ft.; and the corresponding slant pressure is only 3 550 lb. per sq. ft.

It is probable, therefore, that, in view of the strengthening of the original structure, the foundation soil under the higher portions of the structure will not be subjected to loads in excess of 4 000 lb. per sq. ft.

In the design of the footings of the new spillway section, between Buttresses 10 and 19, the fact that the footings rest directly on shale bed-rock made the problem comparatively simple, and detailed discussion appears to be unwarranted. In that section the maximum unit compressive load is approximately 5 tons per sq. ft., whereas, in the writer's opinion, the foundation shale can take at least 20 tons per sq. ft. in compression.

Strength of Footings.—Apparently, it was intended by the designers of the original structure that the load should be transmitted

into the foundation soil through the buttress footings only. Throughout the larger part of the structure the buttress footings are 7 ft. 6 in. wide. The intervening space, also 7 ft. 6 in. wide, between buttress footings, was apparently intended to be simply floored over with reinforced concrete, varying from 8 to 12 in. in thickness. Such flooring, however, was connected rigidly by reinforcing steel to the buttress footings, the heavier lower fiber reinforcement being cut at the center of each bay. In the study of the strength of these footings, in connection with the safe bearing value of the foundation soil, alternatives submitted themselves, neither of which afforded satisfactory results.

In the first place, if the buttress footings were considered to carry the entire vertical load, then manifestly the unit loads would be twice as great as those derived on the assumption that the footings are continuous across the entire 15-ft. width of the bays. For instance, referring again to the typical sections of Plate XII, such doubled maximum vertical soil pressure would become approximately 7 120 lb. per sq. ft., and the maximum slant pressure would become approximately 12 000 lb. per sq. ft. at the down-stream edge of the footings of the original structure under the "most severe conditions within the limits of reason". In the case of the strengthened typical section, such doubled maximum vertical and slant soil pressures would become approximately 6 090 and 8 190 lb. per sq. ft., respectively, under the "most severe conditions". The latter pressures, however, unlike those previously stated, are found at the up-stream edge of the original toe; for it is hardly conceivable that the new toe-wall, which is continuous for the sections of maximum height between old and new spillways, could act otherwise than as a continuous footing, in so far as the transmission of vertical load into the foundation soil is concerned.

The original footings, if assumed to extend only 3 ft. beyond each buttress, were approximately strong enough to carry the doubled unit loads just mentioned, but the adopted safe bearing value of the soil, *viz.*, approximately 5 000 lb. per sq. ft., would evidently be exceeded. Hence, it was imperative that the vertical load on the foundation soil should be distributed over a width of 15 ft., and not merely 7 ft. 6 in., for each buttress.

Turning then to the second or alternative assumption, it is readily seen that only by the use of continuous footings was it possible to keep the loads on the foundation soil within safe limits. As a matter of fact, even in the original structure, some load must necessarily have been carried by the intermediate flooring, inasmuch as it was strictly a part of the adjacent buttress footings. The main buttress footings could not take load (and hence deflect, by reason of compression of the underlying soil) without transmitting part of such load by shear into the adjacent flooring. It is probable, however, that in the downstream portions of the original footings such action caused stresses considerably in excess of the safe working stresses for concrete and steel.

Moreover, in the original portion of the structure, continuity of footings was necessary in order that the footings might act in harmony with the continuous horizontal, or upper, member of the new anchoring wall at the toe. The original footings, however, were not strong enough for this purpose. The weakness lay, not merely in the intermediate reinforced concrete flooring, but also in the main buttress footings themselves, for, under these conditions, the length of the cantilever (measured from the face of the buttress toward the center of the bay) becomes 6 ft. 9 in., instead of 3 ft., as under the former alternative assumption. Thus, in the case of the typical section (Plate XII) the allowable vertical soil pressure on the footings, with assumed working stresses of 700 lb. per sq. in. for the concrete and 18 000 lb. per sq. in. for the reinforcing steel, would be only about 1 700 lb. per sq. ft.

The adopted method of strengthening the footings is shown in the typical sections of Plate XII. It was accomplished by adding both concrete and reinforcing steel. The object was to enable the original intermediate flooring to act as a part of the main footings, which would thereby become continuous, and also to strengthen the original buttress footings. In view of the relatively short clear span of such continuous footings (about 13 ft. 6 in.), it appears reasonably certain that in transmitting the load into the foundation soil the footings act in part as inverted arches. It seemed proper to utilize such arch action in the design, and therefore the arbitrary, but not unreasonable, assumption was made that one-third of the load would be carried by the footings acting as arches and the remainder by the footings

acting as continuous slabs or beams. Such continuous beams have tension in the lower fibers at the buttresses, but have compression in the lower fibers at the centers of the bays.

Concrete was added on top of the original footings adjoining the bases of the buttresses, so as to make more effective the existing steel in the bottom of the footings; whereas, near the middle of the bays, the new concrete serves as a matrix for the new transverse reinforcing steel (bent as shown in section *A-A* of Plate XII). The new concrete was well tied and bound into the old concrete, both by steel dowels extending through the base of each buttress and into the original buttress footings, and also by the roughening of all surfaces of contact. This roughening was done partly with picks, by hand, but mainly by the use of air-hammers. After roughening, the surfaces were thoroughly cleaned and made wet before applying the cement grout coating which immediately preceded the placing of the new concrete.

The new footing concrete, however, has two other functions: First, it serves to embed the tie-steel leading down stream from the anchoring wall at the heel, and hence it was extended to the rear of the bays, even though there the original footings did not require reinforcement. Secondly, it serves to transmit the horizontal thrust from the buttresses and buttress footings into the tie-steel.

The approximately triangular section of concrete, shown in Section *B-B* of Plate XII, at the junction between deck and footings, has been referred to in connection with the description of the new anchoring wall at the heel, serving as it does to take up the reaction of the upper cantilever member of the "heel". The concrete of this triangular section was usually poured in connection with the new "heel", and prior to the placing of the new footing concrete.

At Bays 27 to 33, inclusive, extra depths of new concrete were added over the original footings. This was done partly in order to add strength by adding weight to these sections of maximum height. The quantities of concrete involved, however, were so great that, for the sake of economy, it was deemed advisable to leave a number of large pockets in the concrete. These pockets were made practically water-tight, and act as tanks. From the time that these pockets were first filled with water, they have remained full. Thus the only sacrifice in weight is that due to the difference between the specific gravity of water and concrete.

Another reason for adding such extra concrete was to raise the top of the strengthened footings at least 5 ft. above the bottom of the new outlet channel (shown by Fig. 17). The outlets from the sluice-gates in Bays 30 and 31 were covered with a substantial reinforced concrete slab which completely separates the gate openings and

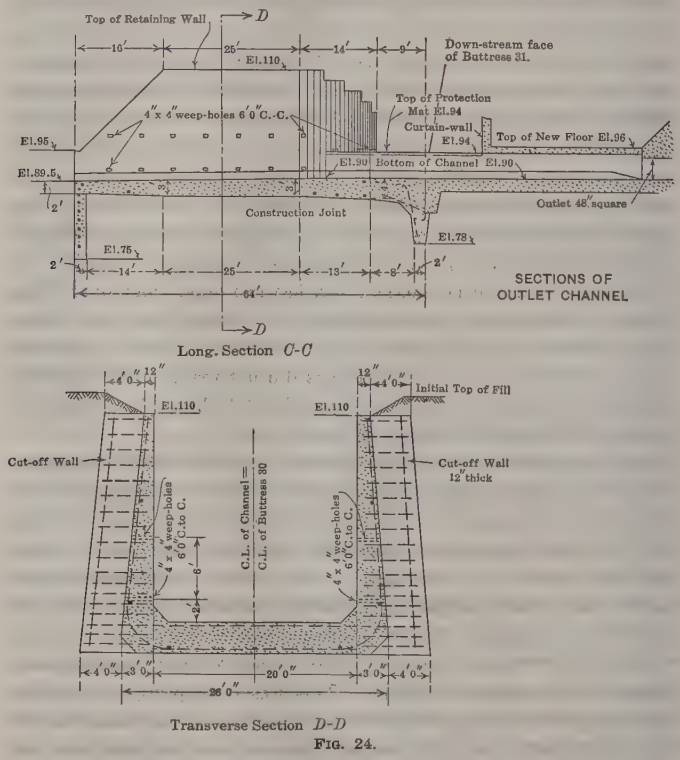


FIG. 24.

outlets from the operating mechanism. Consequently, no spray due to the opening of the gates can interfere with their operation, by freezing in winter, for instance. Moreover, it was thus possible to house in the operating mechanism of the gates entirely by carrying the slab (acting as a roof over the outlets) forward to meet the curtain-wall, as shown in the longitudinal sections of Fig. 24 and Plate XIII.

At the east end of the dam, beginning at Buttress 49, it was considered allowable, as already explained, to increase considerably the unit loads on the foundation soil. Hence, in that section, the original scheme was followed of utilizing the main buttress footings to carry the entire vertical load. For this purpose the original footing required only minor strengthening. Here the possible cracking of the intermediate flooring was considered to be of minor consequence; however, the tie-steel from the new "heel", extending as it does a comparatively short distance down stream from the "heel", was well covered with concrete, doweled both into the floor and the buttress footings.

Pressure Grouting Under Footings.—The contact between the original footings and the foundation soil was in some places found to be faulty. Generally speaking, the original footings had not been carried into the foundation soil as deeply as might have been desired, in consequence of which certain soft spots required compacting. Also, hollow spaces were found under the footings around the 4 by 12-in. weep-holes which had been left at intervals in the footings as constructed originally. Such voids were probably due either to leakage from the reservoir or to flowing ground-water.

Moreover, it was discovered that, at least in some bays, the foundation soil had been washed away at the junction between the footings and the cut-off wall. This is shown clearly in Fig. 23 where the chalk line on the down-stream face of the original cut-off wall shows the extent of the erosion which had taken place. The photograph from which Fig. 23 has been reproduced was taken in Bay 19, but it was evident, from the conditions exposed during the wrecking of adjacent portions of the original structure, that the void in question had no connection with the break in the dam; instead, it probably communicated with a near-by weep-hole through the footings. It is believed, therefore, that this condition did not arise at the time of the failure, but had been brought about gradually by leakage through the unbonded construction joint at the top of the original cut-off wall. The construction joint would naturally open somewhat, due to the deflection of the foundation soil when loaded.

These faulty conditions, it was concluded, could best be remedied by pressure grouting. For this purpose, a Ransome-Caniff grout mixer, of about 4½ cu. ft. capacity, was used; air under pressure was obtained from a small compressing outfit, and utilized at pressures varying from

25 to 100 lb. per sq. in. at the grout mixer. The mixer, in turn, was connected to pipes sealed into the weep-holes with concrete or cement grout. Mixtures of cement and sand, in the proportions of 1:1 or 2:1, were used as far as possible, but, for the greater part of the work, neat cement grout was found to be more practical; the presence of sand in the grout was frequently the cause of clogging the pipe connections and consequent delay.

It was necessary to plug temporarily all weep-holes in the neighborhood of that to which grout was being applied under pressure. Observations showed that grout under pressure traveled under the footings for a distance of at least 100 ft. Inasmuch as the water contained in the cement grout helped to fill the voids, it was found necessary to adopt the process of forcing grout into the voids connecting with a given weep-hole until they refused to accept more, and then suspending grouting for a day or more. In the meantime the grout would set, leaving the uncombined water free to be blown from under the footings, on subsequent resumption of grouting, by simply unplugging temporarily a few communicating weep-holes. Such holes were then re-plugged and more grout was forced into the voids. This process was continued for each weep-hole until finally it was found that the hole would accept no more grout. After grouting had been completed and the grout had set, the weep-holes were again opened, so as to communicate with the underlying soil, by simply punching or drilling holes through the grout.

By these means, it is believed, the foundation soil under the original footings was thoroughly compacted so as to warrant the assumption that the footings are everywhere in contact with the foundation soil, either directly or through intermediate grout.

Leakage and Drainage.

Leakage Through Foundation Soil.—Reference has been made to the leakage which took place through the construction joint at the top of the original cut-off wall, and also to the fact that this trouble was remedied as an incidental result of the construction of the new anchoring wall at the heel. Further leakage, however, was discovered under and through the original cut-off wall, especially where the structure is of maximum height. During the work of investigation, prior to beginning actual reconstruction, test-pits were sunk up stream from.

and adjacent to, the original cut-off wall at Buttresses *B* and 37 (Plate IX). The pit at Buttress *B* was approximately 45 ft. deep, and by means of it the cut-off was investigated at its deepest section. In neither of these test pits was the cut-off wall found to be porous or the seal into bed-rock defective.

Except for a test pit sunk with great difficulty through the footings and foundation soil adjacent to the down-stream side of the cut-off wall at Bay 28, the investigation of the cut-off at the sections of maximum height was postponed until the actual work of reconstruction was being carried on. The cut-off in this region, *viz.*, between Buttresses 26 and 32, was relatively shallow, at least as compared with the hydrostatic head due to the reservoir; and therefore it was the more important that this portion of the cut-off should be impervious. The postponement of the investigation in this region was due only to the great expense of putting down test pits so near the river bed without adequate coffer-damming.

In connection with the excavation for the new "heel", therefore, test pits were sunk at intervals, and extending from the elevation of the bottom of the "heel" down to the bottom of the original cut-off. The findings in the test pit sunk through the footings in Bay 28 had already indicated, much to the writer's surprise, that the cut-off in that region was not all that it should be, and the test pits sunk from the trench excavation for the "heel" on the up-stream side of the cut-off confirmed these findings. It was definitely established that there was easy communication for water between the up-stream and down-stream sides of the cut-off wall at Bay 28. Probably some of the water which flowed back and forth, depending on the hydrostatic pressure conditions, passed through the cut-off wall itself; but, at this point, undoubtedly the greater part of the communication was underneath, through openings between the cut-off wall and the bed-rock.

At Bay 30, also, it was evident that the seal of the cut-off into bed-rock was imperfect. In fact, at one point a layer of sandy mud, averaging $\frac{1}{4}$ in. thick, was plainly visible between the concrete and the shale, indicating that, in the original construction, the trench had not been properly cleaned preparatory to the placing of concrete. Moreover, the cut-off wall itself was porous to the extent of allowing a constant trickle of water to pass through from the down-stream side. That there was any such leakage at all was the more noteworthy because

of the fact that the hydrostatic pressure on the down-stream side was due solely to the ground-water in the clayey foundation soil—though it should be remembered, of course, that the foundation soil under this maximum height portion of the dam is of poorer quality than that at any other part of the structure.

In view of the foregoing findings, it was determined to underpin the original cut-off wall for approximately 200 ft., *viz.*, from about Buttress 32 westward to the new spillway. A notch, about 24 in. deep and 18 in. wide, was cut into the bed-rock, with its center approximately at the up-stream edge of the original cut-off wall. The notch was carefully cleaned and kept free from water while it was being filled with a rich concrete which was carried to a height of at least 12 in. above the bottom of the old cut-off.

This work proved to be the most expensive part of the entire reconstruction, partly because of the constant fight with the water which flowed into the trenches through the pervious soil under the river-bed, and partly because of the fact that the character of the soil required practically continuous sheeting and bracing. So difficult, in fact, was it to carry on the work and to hold the up-stream side of the excavation for the "heel" that at Bays 30 and 31 it was found advisable first to pour the "heel" concrete and then, after that concrete had set, to tunnel under it in order to investigate the efficacy of the cut-off seal into the rock. Such tunneling was done from the adjacent bays, and, to insure solid work under the "heel" at Bays 30 and 31, grout pipes were inserted while the concrete for the cut-off underpinning was being placed, the ends of the pipes being exposed under these portions of the "heel". Later, the voids at the ends of these pipes were filled with grout, by using the Ransome-Caniff grout mixer, under a pressure of about 75 lb. per sq. in.

At certain places, as shown on Plate IX, the cut-off underpinning concrete was extended on the up-stream side of the cut-off up to the elevation of the bottom of the "heel". In such cases a slip joint was made between the "heel" and the underpinning concrete. This joint consisted essentially of a longitudinal groove, about 12 in. wide and 6 in. deep, lined with tar-paper. The groove was centered on the bottom of the "heel". When the "heel" concrete was poured, therefore, a tongue of concrete, likewise 12 in. wide and 6 in. deep, was formed in the groove integral with the "heel" concrete. On all horizontal

surfaces, and on the up-stream vertical face of the groove, single sheets of two-ply tar-paper were used to separate the two bodies of concrete. On the down-stream vertical face of the groove, however, three sheets of two-ply tar-paper were used.

The purpose of the slip joint is to allow the "heel" to move down stream a fraction of an inch under the horizontal thrust acting against the dam and hence transmitted by the tie-steel into the "heel". Without such "play" an unfavorable loading might be caused at the lower end of the long cantilever member of the "heel", because of the resistance of the relatively inelastic underpinning concrete. The latter, of course, is practically immovable because of its bond into the bed-rock. The utility of the slip joint is probably to be justified on theoretical rather than practical grounds, but, in view of the small expense involved, there seemed to be no warrant for omitting this precaution.

At about the middle of Bay 26 it was found that there was a pervious spot in the bed-rock strata just below the practicable limit of underpinning. Therefore a churn-drill hole was sunk into the strata in question, and grout pipe connections were made, which, after the completion of the underpinning and the new "heel", were utilized in an attempt to force grout into the stratum. However, a pressure of 75 lb. per sq. in. did not suffice to force down an appreciable quantity of grout.

It is not improbable that the failure to force more grout into this stratum was due to the fact that there was no vent for such rock-water as might have been displaced by grout had a means of escape for such water been provided. It happened that, for the grouting of the underpinning of Bays 30 and 31, as previously mentioned, two grout pipe connections had been left, and thereafter on the work, profiting by the experience at Bay 26, at least two grout pipes were placed in each void to be filled. Usually, the second pipe was connected to the pressure grouting apparatus only after the first pipe refused to accept any more grout. In such cases it was commonly impossible to force grout into the second pipe, and apparently the main function of that pipe was to serve as a vent. In the writer's opinion, it is not unlikely that some of the reported failures of pressure grouting operations at other dam sites have been due to the fact that no such vent existed, or had been provided, for the escape

of air or ground-water contained in the voids sought to be filled by grouting.

Referring again to the faults in the original cut-off construction, which required remedying by the means just described, the reconstruction afforded a rare opportunity for observing the results of this construction work, which had probably been done with average care. These observations show that great care is necessary even in foundation work, where it is too often assumed that any kind of work will suffice.

Extension of Cut-Off.—Inasmuch as the insufficient depth of cut-off wall in the neighborhood of the break in the dam was undoubtedly the direct cause of the failure, it was advisable to pay especially careful attention to any further deficiency in depth of cut-off. At the sections of maximum height, precautions were taken in the way of underpinning the original cut-off wall, as discussed previously. Such underpinning extended westward to the new spillway which in turn covered completely the area affected by the failure. (Plate XI.) Here the combination cut-off and anchoring wall at the heel extends into the shale rock to an average depth of about 10 ft., in the manner illustrated on Plate X. The depth is greatest in the neighborhood of Buttresses 14 and 15, because there, as mentioned previously, the rock was found to be somewhat shattered.

In the same region the rock yielded a considerable quantity of water, especially after the drain holes had been drilled. Consequently, after the combination cut-off and anchoring wall had been completed, a hole was drilled, with the Calyx machine, through a pipe which had been embedded in the center of the wall at about the middle of Bay 13. The hole penetrated the underlying shale to about Elevation 70. It was intended to grout this hole for the purpose of cutting off any communication with the reservoir which might exist in this neighborhood through passages in the rock under the cut-off. It was assumed that the existence of any such connection would be shown, when pressure grouting was applied, through the manifestation of bubbling or other symptoms at a drain hole extending to about the same elevation and approximately 7 ft. down stream in Bay 13. However, a surprise was afforded, in that the hole would accept no grout, and it was apparent that the rock-water passages, which had been tapped a short distance down stream in Bay 13, either came from a greater depth, or more

probably communicated to the westward—the slope of the bed-rock (Plate IX) being upward and westward.

West of the new spillway, beginning at Buttress 10, the depth of the original cut-off was as inadequate as at the point of failure. At some places, especially near steps in the bottom, the depth of the cut-off was less than 5 ft. below the bottom of the footings (Plate IX). It is probable that everywhere in this region, except possibly at one or two of such steps, the original cut-off extended to a depth of at least 6 ft. below the original ground surface. That the wall was not deep enough to cut off a pervious stratum of dark color in the overburden will be seen by referring to Fig. 7, in the upper right corner of which the bottom of the original cut-off wall is shown supported by a prop.

The entire west end of the dam, *viz.*, between Buttresses 1 and 10, was provided with a new "heel", the required depth of which varied from 1 to 5 ft. below the bottom of the original cut-off. The new "heel" was made not less than 2 ft. 6 in. wide at the bottom, this being a width convenient for trench excavation. A cut-off wall of this minimum width was then extended downward from the bottom of the "heel" to seal into the bed-rock. In this region the bed-rock consists of shale of a very good quality; in fact, in some places the rock is so hard that a notch only 6 in. deep was considered to afford a sufficiently good seal. At other places, however, the notching was carried to a depth of more than 3 ft. on account of penetrating a stratum of soft shale which in part overlies the very hard stratum.

At the bottom of the required depth of "heel" a standard slip joint was provided, in the manner previously described; and just under the old cut-off (but above the slip joint) a shoulder of concrete, with a minimum width of 12 in., was formed. The purpose of such a shoulder of concrete was to make certain that any vertical load transmitted into the old cut-off wall would in turn be taken up by the new "heel" and cut-off, thus guarding against any possible separation of the old concrete from the new and against the placing of any excessive shearing stress on the tie-steel connecting the new anchoring wall to the original structure.

As will be seen from Plate IX, a core-wall had been provided in the original construction extending 30 ft. west of Buttress 1. As a part of the reconstruction, this core-wall was extended upward to

the elevation of the new parapet (Elevation 142.5) and downward to seal into the shale. Also, because of the increase in assumed high water elevation, it was extended about 15 ft. farther westward. The precautions taken in providing a cut-off against percolation at the west end of the dam were of a more conservative nature, for the reason that it was near this end of the dam that the failure had occurred.

Between Buttresses 32 and 46 the depth of the original cut-off, with a maximum of 45 ft., was concluded to be sufficient in proportion to the maximum hydrostatic pressure. This conclusion was based on the fairly well substantiated assumption that in this region the original cut-off is impervious throughout its entire depth, but it did not depend on the character of the seal of the cut-off into the bed-rock. As a matter of fact, however, a personal inspection of the bond between concrete and rock as exposed by the test pits at Buttresses *B* and 37 convinced the writer that the seal is as effective as could be desired. East of Buttress 46, on the other hand, certain difficulties were encountered.

It had been assumed by the builders of the original structure that the cut-off extended to bed-rock at least as far east as the approximately vertical step in the cut-off at the middle of Bay 54 (Plate IX). In fact, all records of the original work show the cut-off as being in contact with bed-rock. It is not surprising, therefore, that the results of core-drill holes Nos. 3 and 8, sunk during the investigation prior to reconstruction, were discredited when they indicated that bed-rock was reached only at a considerable distance below the bottom of the original cut-off. Nevertheless, some mental reservations were made, and subsequently a test pit was sunk at the middle of Bay 48. The findings were by no means comforting, for, after penetrating flat-top boulders, the upper surfaces of which offered some reason for their previously erroneous identification as bed-rock, the pit extended through clay deposits into strata of disintegrated coal. Rock was not reached until at about Elevation 92. It is a shale, and quite satisfactory.

In view of the discovery of the disintegrated coal, which was decidedly pervious, the existing cut-off was manifestly inadequate. This was especially true of the portion east of the middle of Bay 48. It was concluded, therefore, to underpin the original cut-off down to a line sloping with approximate uniformity from Elevation 92 at Buttress 46 to Elevation 108 at Buttress 53, and meeting the bottom of the original

cut-off at the latter point. The proviso was made, however, that the disintegrated coal should be cut off wherever its top came within 1 ft. below the bottom of such sloping line. The probable genesis of the disintegrated coal and the measures taken to cut it off are shown on Plate IX. Drill holes Nos. 9 and 10 were sunk after the discovery of the disturbing conditions just described.

It was also concluded to underpin the original cut-off east of the middle of Bay 54, as well as the core-wall which originally extended 50 ft. east of Buttress 56. Excavation for the underpinning in these two localities proved to be very difficult because of the existence of a sandstone talus in the form of huge boulders. The most economical method was found to be the sinking of shafts from the surface of the ground and the construction of tunnel drifts leading from such shafts under the original cut-off. Some of the crevices discovered between the boulders were found to be filled with clay, and others were open. In one instance a crevice at least 2 in. wide was traced for 6 ft., and it extended even beyond that distance. The shattered rock was very difficult to excavate, and required frequent blasting.

Before the excavation was filled with concrete, grout holes were churn-drilled into the rock, and grout pipes were placed in crevices so that later, when these were pressure-grouted, it is probable that an impervious rock, concrete, and grout curtain was formed, extending between Buttresses 54 and 56 down to at least Elevation 110. Similarly, grout pipes were left in the higher portions of the tunnel drifts so as to insure, later, the filling, by pressure grouting, of any air pockets left in the higher sections. The same grout pipes served to allow the escape of confined air while the main concrete was being placed. The concrete was carried to a considerable height in the shafts previously mentioned and, because of its slushy consistency, undoubtedly created considerable pressure throughout the drifts. Pressure-grouting was not applied until the concrete had set for a day. The east core-wall was extended approximately 24 ft. farther east. Both the old and new portions of the core-wall were built up to Elevation 142.5, to correspond with the top of the new parapet.

Drainage.—In the original structure the intermediate flooring of each bay had 4 by 12-in. weep-holes. A number of these holes existed at each edge of the main buttress footings, the shorter dimension of the holes being parallel to the footings. The weep-holes were in

pairs, one hole at each edge of the intermediate flooring; the pairs were about 8 ft. from center to center, though the spacing was closer at the up-stream ends of the bays. As a matter of fact, a considerable number of these weep-holes were found to be clogged with concrete which had evidently flowed under the weep-hole forms at the time the original footing or flooring concrete was being placed. Apparently, it had not been thought necessary to provide deeper drainage for the foundation soil.

In planning the reconstruction of the dam, however, it was concluded to provide more positive means of drainage. The system adopted for that portion of the original structure which remained intact differed from that adopted for the new spillway section:

1.—Drainage of Foundation Soil.—In the preliminary plans for reconstruction, the writer proceeded on the theory that the more drains the better, and especially the more positive, the relief of uplift pressure. Consequently, it was proposed in general to extend three drains per bay into the foundation soil. These drains were to consist of perforated pipes such as used in the system of drainage finally adopted. The up-stream drains were to be inclined, and thus were to extend to within about 5 ft. of the down-stream face of the cut-off wall and to a depth of about 5 ft. below the bottom of the new "heel". The middle and down-stream drains of each bay were to be vertical, at distances of about 20 and 30 ft., respectively, from the cut-off wall. At the seven bays of maximum height it was proposed to place the up-stream drains 7½ ft. from center to center transversely, thus providing two up-stream drains per bay. It was also proposed to keep all the original weep-holes open through the original concrete and the concrete of the footing strengthening.

On the other hand, the system of drainage finally adopted, though utilizing the same methods as proposed in the preliminary plans, differed as to the number and location of drains. It was based on two main considerations, namely:

(*a*) The necessity of relieving serious uplift pressure; but, also,
(*b*) The inadvisability of facilitating percolation through the foundation soil.

It was concluded that deep drains might be harmful in earth foundations under circumstances where they would not be harmful

in rock foundations, because, in the former, they might actually facilitate the flow of water by providing artificial channels where previously there was no channel at all. Especially might such results follow if the deep drains were to extend close to the bottom of the cut-off wall.

A consideration of secondary importance was the effect of the low values of the coefficient of frictional resistance assumed in the reconstruction design. Manifestly, a given uplift pressure reduces the margin of safety against sliding by a less amount where there is a low coefficient of frictional resistance than it does where the coefficient is relatively high. It was believed to be preferable, therefore, to take such risk as might result from the existence of pockets, as it were, of unrelieved uplift pressure near the down-stream side of the cut-off wall rather than to invite further leakage by tapping such relatively harmless, deep-lying pockets of water under pressure.

In consequence of these considerations, a system of drainage was adopted which involved but a single deep drain per bay for the higher sections of the intact portions of the original structure, resting as they do on clayey foundation soil. The positions of the drains are indicated in the typical sections of Plates IX and XII. From the former it is seen that the deep drains for the foundation soil extend from Bays 20 to 47, inclusive. The drains consist of 3-in. wrought-iron pipes, perforated with four $\frac{3}{4}$-in. holes per linear foot, one hole in each quadrant. Pipe drains, rather than unlined holes, were adopted for the foundation soil, in order to make the drains relatively permanent, and also because unlined openings would not resist erosion of the foundation soil by leakage water.

The deep drains are about 20 ft. down stream from the cut-off wall, except at the bays of lesser height, where they are at a minimum distance of 12 ft. from it. The drains ordinarily extend to a depth of 5 ft. below the bottom of the new "heel", though they were limited to a depth equalling two-thirds of that of the cut-off, measured from the bottom of the original footing to the bottom of the underpinning, if any, of the original cut-off. They were sunk in lengths of 12 ft. or less. Flush-joint piping was used, so as to avoid couplings or other projections; the latter might leave an annular space around the drains to serve as an outside channel for flowing water without the protection of a metal lining.

Great difficulty was experienced in sinking the drain pipes, primarily because of the boulders encountered in the foundation soil. Various methods were tried, among them the process of jetting, that is, of sinking a drain pipe, open at the lower end, by using a jet of water under pressure playing on the soil inside the pipe near its lower end, so as to allow the pipe to be forced down by hammer blows at the top.

The most satisfactory method, however, and that generally adopted, involved the use of a pile-hammer. For this purpose a McKiernan-Terry, No. 2, pile-hammer was placed on the job and operated either by air or by steam at a pressure of about 80 lb. per sq. in. In order to facilitate driving, the blacksmith on the work fitted the drain pipes with solid points made from pieces of old steel shafting, approximately 3 in. in diameter. The points were welded inside the ends of the pipes, with results as shown in Fig. 19. This figure shows also the perforations in the pipes. It is to be admitted that the perforations were probably clogged with clay to a certain extent as the pipes were sunk. On the other hand, any considerable hydrostatic pressure near the drain pipes would soon open the perforations.

A simple wooden cap prevented injury to the pipe, and the flush-joint pipe did not prove so weak at the joints as to buckle or otherwise cause trouble. In a number of instances, however, boulders proved to be impassable obstacles, and in such cases pipes were driven at near-by locations. In each instance the drain pipes were sunk through the weep-holes of the original structure. The process was necessarily slow. Sometimes it required 12 hours, and even 24 hours, to drive a pipe to a depth of 20 ft.

In general, the deep drain pipes were encased in grout or concrete through the footings and for a depth of about 18 in. below the footings, with the object of keeping the water percolating through the deeper foundation soil separate from leakage water manifesting itself immediately under the footings, and finding its origin perhaps through the cut-off wall. Thus the deep drains are enabled the better to perform their function as detectives, so to speak. The upper ends of the drain pipes were threaded and left exposed for at least 3 in., so as to allow the pipes to be capped or to be connected to pressure gauges. The pipes, of course, were thoroughly cleaned out after they were driven. Very few of the deep drains showed leakage after the work of reconstruction was completed, and none showed any considerable leakage.

Further, under the adopted system of drainage, the two up-stream pairs of weep-holes in each bay were plugged with concrete in connection with the pouring of the triangular section of concrete at the junction between deck and footings. About one-half of the remaining weep-holes—distributed as uniformly as practicable—were left open.

As the reservoir refilled during the latter part of the work of reconstruction, leakage appeared at weep-holes in Bays 51 and 36. At first it was thought that possibly the leakage at Bay 51 was caused by ground-water, perhaps from the water being fed into drill hole 10 (Plate IX) which was being sunk at the time. However, by coloring the reservoir water immediately up stream from Bay 51 with potassium permanganate, it was finally proved that the leakage came through the cut-off. Probably it was due to inadequate bond along a horizontal construction joint in the new "heel", within a foot or two in elevation of the old construction joint at the top of the cut-off wall. The leakage was practically stopped by the simple expedient of dumping clay into the reservoir over the new "heel" from a float extending out from the east shore.

It is probable that in time the leak at Bay 51 would have sealed itself, as has since become true of the leak at Bay 36. In the latter instance the same remedy was applied as at Bay 51, and fine cinders, as well as clay, were dumped from boats into the reservoir water over the "heel". The cure, however, was not effected in this manner; but it was noted that the leakage was gradually decreasing, even though the water level of the reservoir grew higher, until by November 1st, 1915, there was hardly any leakage at this point.

If the leakage at these two points was due to carelessly treated construction joints in the new "heel", the stopping of the leakage may in turn have been due—in part, at least—to the taking of some of the horizontal thrust by the new "heel", with consequent tightening of the construction joint by reason of the compression caused in the up-stream portion of the "heel".

Other leaks, of a minor nature, which appeared in the original portions of the structure have stopped or are stopping themselves in the same manner. Thus on May 25th, 1915, there were eighteen leaks or traces of leakage from weep-holes or drain pipes in the foundation soil, whereas, by November 1st, 1915, there were only three, and these the attendant designated as "traces". In no case

has a pressure test indicated a head of more than 2 ft. above the tops of the weep-holes or drain pipes.

At times certain of the drainage openings yielded muddy water, instead of the normally clear water. In the opinion of the writer, these phenomena, usually temporary, were due to adjustments in the foundation soil caused by increases in loading as the reservoir refilled. In a few instances such openings were grouted shut, rather than take any chances of harmful erosion under the footings.

It may not be amiss to refer here to another phenomenon, namely, that in a number of pipes which are sealed into weep-holes and communicate through the footings into the foundation soil, the water level remains practically at the top of the pipes, which are about 2 in. in diameter and extend about 6 in. above the top of the concrete footings. These pipes do not actually weep, except in the sense that when water is dipped out of the pipes, they refill in a day or two, but again do not overflow. The possibility is indicated that capillary tension over the surface of the ground-water rising in the pipes is sufficient to prevent overflow, but the writer has been unable to advance any explanation which is satisfactory to himself.

2.—Drainage of Rock Foundation at New Spillway.—In the case of the shale forming the foundation for the buttress footings of the new spillway, there is, of course, no danger that deep drains will facilitate erosion of the foundation. This is especially true because of the fact that the shale contains practically no lime which might be taken out in chemical solution. Moreover, as described in detail under the heading "Geological and Foundation Conditions", the rock contains water-bearing seams, and it was possible that ground-water communication might exist between the reservoir and certain springs which appeared in the area excavated for the new spillway footings. Hence it was concluded to provide more drains at the new spillway than in the foundation soil under the intact portions of the original structure.

The system of drainage provided at the new spillway consists in general of three deep drains per bay, placed as shown in the typical sections of Plate X. The drains were sunk with the Calyx, shot-drilling apparatus. In the bed-rock they remain unlined. Above the shale, however, 4-in. pipes were sealed as well as possible into the rock and carried up at least to Elevation 96, in order that leakage

might be detected above the level of the water which ordinarily stands in the new spillway bays as high as the elevation of the horizontal drain through the foot of the spillway apron (Plate X). It was practically impossible to prevent the continuously flooded condition of the space between the new spillway footings; at the same time, this condition has the merit of preventing disintegration of any shale by exposure to the atmosphere.

The up-stream drain of each trio extends down to Elevation 70, well below the bottom of the combination cut-off and anchoring wall. The middle and down-stream drains extend down only to Elevation 75, which elevation also is below that of the bottom of the cut-off. The number of drains was increased at Bays 11 and 12, being the two most westwardly bays of the new spillway; and, inasmuch as the strata slope upward toward the west bank, it is probable that the rock-water, following the dip of the strata, will be intercepted at these bays.

Ever since the deep drains of the new spillway were sunk there has been a flow of rock-water in Bays 12, 13, 16, and 19. However, both the quality of the water and its relatively constant yield prove that there is little, if any, direct communication between such rock-water and the reservoir. The reservoir pressure doubtless has some influence on the rock-water in question, for there was a minor increase in the yield as the reservoir gradually refilled. Yet it is also true that during "dry" periods the rock-water yield has decreased somewhat, even though during the same periods there was but little fluctuation in reservoir level. The maximum rock-water pressure, measured at the tops of the new spillway drains, was equivalent to a head of about 2 ft. The drains overflow freely into the surrounding pools of water.

In view of the drainage system provided throughout the higher portions of the structure, it appears reasonable to assume that, if the drains are not frozen shut at times when uplift pressure may be accumulating under the footings, no serious uplift pressure is to be feared. The assumptions of uplift pressure under the "most severe conditions within the limits of reason", as set forth in Table 3, are based on the drainage provision above described.

Curtain-Wall and Roof.—It was primarily to prevent any considerable freezing at the top of the weep-holes and deep drains throughout the higher bulkhead sections of the dam that a curtain-wall and roof were constructed, as illustrated in the typical section *B-B* of Plate XII.

Manifestly, an ice plug, say, 6 in. deep, at the top of a weep-hole or drain pipe, would for practical purposes serve to confine water under the footings as effectively as though the openings had been filled with concrete and made a part of the footings.

It is believed that, as the result of housing in all bays having deep drains, namely, Bays 11 to 47, inclusive (Figs. 16 and 17), the temperature of the corresponding footings will be prevented from falling to more than a few degrees below the freezing point. During cold weather the radiation of heat outward through the curtain-wall and roof will be largely counterbalanced by the radiation upward of heat from the unfrozen foundation soil and by the further radiation of heat from the reservoir water through the deck of the dam into the enclosed space. Water is at its greatest density at about 39° Fahr., with the density gradually decreasing as the temperature decreases to the freezing point. Consequently, water in the reservoir under an ice covering is at least several degrees above freezing temperature; the difference may be as great as 7 degrees. The assumption made as to the effect of the curtain-wall and roof is supported by observations said to have been made at various hollow dams which are provided with both deck and apron, thus being entirely enclosed. It is reported that at some such structures freezing does not occur in the enclosed portions.

As soon as practicable in the late winter of 1914 the writer had maximum and minimum thermometers placed in the old spillway section of the Stony River Dam, and had the larger openings into that section closed so as to allow a series of records to be made, covering exterior and interior temperatures. Conditions were not favorable for conclusive results, because the reservoir, of course, had been emptied at the time of failure, thus exposing both apron and deck to the cold, with the result that the only source of heat was the foundation soil under the old spillway. Nevertheless, even these observations showed clearly that variations of temperature inside the enclosed spillway section were far less than the corresponding variations in outside temperature; also, that periods of relatively higher and lower temperature inside the structure lagged considerably behind the corresponding periods of relatively higher and lower temperature outside.

During the winter of 1915-16 the lowest temperature within the enclosed portions of the dam was + 26° Fahr., and the lowest exterior

temperature was — 12° Fahr. Under these conditions ice formation has taken place, but not of a serious nature. At the very least, the enclosing of the bays enables the attendant to inspect carefully the drainage system, regardless of weather conditions, and also protects the operating mechanism of the outlet gates. When ice forms within the enclosed portions, the attendant can readily keep it broken or cut away so as to prevent any harmful results.

A portion of the curtain was constructed in the form of a roof, instead of continuing the wall vertically to intersect the deck, in order to avoid enclosing what would have been a triangular section at the top, exposed to cold air on the down-stream side and frequently to the ice covering of the reservoir on the up-stream side. For the bulkhead section between the old and new spillways, the curtain-wall was made 12 in. and the roof 7 in. thick, and, for the eleven bays immediately east of the old spillway, the thickness of the curtain-wall was reduced to 6 in., the roof remaining 7 in. in thickness. At the base of each curtain-wall there is an opening, about 12 by 18 in., fitted with a wooden flap door, as shown on Plate XII, so as to allow the outward flow of any leakage water, but at the same time to keep air currents and snow out of the enclosed space. Wooden ladders bolted to the buttresses are provided between the walkway and the floor at each enclosed bay.

In addition to this primary purpose, the curtain-wall and roof furnish effective lateral stiffening for the buttresses. The writer does not suggest that the bracing of the buttresses in the original structure was insufficient, yet he confesses that he feels better satisfied with the buttresses braced by the curtain-wall, especially as regards the lower portions of those of maximum height, where originally an additional lower set of horizontal brace-beams might have been desirable.

Furthermore, the curtain-wall tends to equalize deflection in the foundation soil under the footings, and, consequently, to shift the load from weaker to firmer soil. Finally, the curtain-wall was utilized in a minor way in the design of the strengthening of the intermediate flooring, allowing, as it did, the omission of transverse reinforcement for a reasonable distance on each side of it.

Excepting for these secondary functions of the curtain, it would have been well to make the wall and roof hollow, for instance, by the

use of cement plaster on metal lath. A hollow construction could have been obtained with concrete, of course, but at a sacrifice of economy. Hollow construction would have better conserved the heat available for warming the enclosed portions of the bays. Relative to this it is interesting to note that more freezing has taken place in the eleven bays immediately east of the old spillway, where the curtain-wall is only 6 in. thick, than in the remainder of the enclosed bays, where the wall is at least double that thickness.

Leakage in Superstructure.—At the time of failure there were pronounced leaks at certain contraction joints in the deck. In the type of construction used the deck slabs are not in contact with each other at the buttresses. Instead, a tongue, integral with the buttresses and varying from 18 to 24 in. in width, separates the slabs. Contraction joints exist, therefore, between these tongues and the slabs, over all buttress haunches. It appears that, in the original construction, insufficient care was taken in forming these joints, with the result that in most cases they did not fulfill their intended functions, and contraction cracks occurred only at intervals averaging about 75 ft. The tenacity of the bond in certain of the intended joints is well illustrated in Fig. 5, which shows the deck of Bay 18 overhanging westwardly for a distance of about 13 ft., the adjacent Buttresses 17 and 16 with their intermediate deck having fallen away due to the undermining of the buttress footings. Naturally, such joints as did act opened excessively and allowed considerable leakage to take place.

The worst leakage of this kind occurred on the east side of Buttress 29, where, during the severely cold weather obtaining at the time of the failure, ice formed to such an extent as to block the stairway leading from the walkway down to the outlet gate-operating mechanism. It was considered impracticable to attempt to reconstruct all the contraction joints so as to make them perform properly their intended function. Instead, all joints which were acting too freely were made as nearly water-tight as possible by cleaning them out thoroughly, packing them with tar-paper placed on edge, and then pouring in hot asphalt. The asphalt has a tendency to flow from the joints during extremely hot weather, but it is believed that such asphalt as remains in the joints, together with sediment deposited by leakage water, will prevent any serious leakage. As a matter of fact, the joint at Buttress 29 still leaks somewhat, especially in cold weather. However, by

reason of the bay being enclosed by the curtain-wall and roof, the attendant can now with little difficulty clear the stairway of the trifling quantity of ice that forms during severely cold weather.

In the new spillway section precautions were taken to insure positive action of the contraction joints by covering the entire surface of such joints with two thicknesses of three-ply tar-paper, or the equivalent thereof. The leakage here has been inconsequential.

Miscellaneous Problems.

Strength of Decks and Buttresses.—In the case of the greatest flood reasonably to be anticipated, the deck of the original structure would be subjected to greater loading than that for which it was designed. The resulting increase in stresses would be greatest at the bottom of the upper lift of the deck, namely, at Elevation 126. Here the maximum stresses with head-water at Elevation 142.25 would be approximately 22 000 lb. per sq. in. in the reinforcing steel and 800 lb. per sq. in. in the concrete. The original steel is reported to have been purchased under a guaranty of 55 000 lb. per sq. in., minimum elastic limit, and the concrete was of a character warranting the assumption of an ultimate strength of at least 2 000 lb. per sq. in. in 90 days.

Under these conditions, it did not appear necessary or advisable to strengthen the decks of the original structure. Likewise, it was not considered necessary to strengthen the buttresses, other than to give them the benefit of the incidental stiffening provided by the curtain-wall previously described.

At the new spillway section, on the other hand, it was considered proper to design the deck for a loading due to head-water at Elevation 142.25, notwithstanding the fact that the new spillway deck thus has greater strength than the deck of the original portions of the structure. Also, the thickness of the buttresses was increased, as compared with the original structure. Above Elevation 112 the increase in thickness was a minimum of 4 in., and below that elevation a minimum of 6 in. Furthermore, the new buttresses are more heavily braced laterally than in the case of the original construction. The walkway slab is thickened so as to serve as a brace-beam, with its principal dimension horizontal, and two other brace-beams, with their principal dimensions vertical, are provided lower in the structure. Detailed comparisons may be made by examining the data on Plates X and XII.

Increase in Storage Capacity.—In view of the much larger spillway capacity provided in the reconstruction, it appeared logical to make temporary use of some of the depth over the spillway by providing increased storage capacity. It was concluded to be feasible to utilize in this way approximately 3 ft. of the 6 ft. 6 in. available depth over the main spillways. Such use for storage purposes, however, is temporary only in the sense that, during extraordinary floods, the entire depth over the main spillways must be available for flood discharging purposes. The resulting increase in the available storage capacity was approximately 380 000 000 gal., or about 25% (Fig. 10), and it afforded a certain consolation to the owner for the heavy expenditures entailed by the failure and reconstruction of the dam. It may reasonably be said that, even in the case of the original structure, it would have been possible similarly to utilize some of the 3 ft. difference in elevation between the crest of the spillway and that of the bulkhead sections; yet it is equally true that an appreciable quantity of additional storage could have been obtained only at the risk of overtopping the structure during floods.

Flash-Board Supports.—The problem involved in increasing the available storage capacity by about 25% was to provide a temporary dam, in other words, flash-boards, which would fail within comparatively narrow, predetermined limits of loading. Thus, it was imperative that 3-ft. flash-boards should not be overtopped by more than 3 ft. 6 in. at the very maximum. The effect of the use of such 3-ft. flash-boards on the flood discharging capacity of the spillways was carefully considered, and a number of hypothetical cases were worked out. One of these cases, previously discussed in this paper, is shown in Fig. 14, wherein it was assumed that, with the greatest flood reasonably to be provided for, the 3-ft. flash-boards might not fail until the head-water had reached an elevation 5 ft. above the base of the flash-boards. The result of such studies was the conclusion that the use of flash-boards would not affect seriously the maximum flood discharging capacity of the dam as reconstructed.

It was impracticable to provide at small cost reliable means which would cause the flash-boards to fail with head-water 3 ft. over the boards, and yet not fail at lower elevations. On the other hand, it is highly undesirable that the flash-boards should fail under any except the more severe freshets; otherwise the inconvenience of renewing

the flash-boards and of the loss of the 3-ft. depth of stored water would occur too frequently.

Both of these considerations were subservient to a third condition, namely, that the means of lowering the flash-boards or causing them to fail must be absolutely automatic, inasmuch as conditions warranted the employment of only a single attendant, and at the critical time he might be absent or asleep, or might for any other reason not attend to the flash-boards when necessary. Finally, the cost of the flash-board provision and of replacing the flash-boards was necessarily to be made as low as possible.

For the purpose of meeting the foregoing requirements, the writer had about seventy experiments made on steel bars or "pins" to serve as flash-board supports. The experiments were intended chiefly to find a material which would fail at a predetermined stress, but would not allow the pins to bend considerably at lower stresses. It was also desired that, when failure occurred, it should consist in the pins snapping off at approximately the elevation of the spillway crest, thus allowing the crest to be cleared absolutely of all obstruction by the flash-boards and their supports. The experiments were performed in the outlet channel, where, by using the sluice-gates and the previously accumulated head in the partly refilled reservoir, it was possible to duplicate the heads which would exist under actual operating conditions, and hence use flash-boards and supports of the same size as under actual conditions.

A controlling condition was the fact that flash-board sockets had been provided in the original spillway 3 ft. 6 in. from center to center. The same spacing was used for the crest of the new spillway, so as to allow interchange of equipment. The scope of the tests covered various conditions of head, lengths of time during which heads were maintained, and degrees of turbulence (akin to reservoir wave action), as well as various kinds and conditions of steel. Inasmuch as it was desired to avoid the use of pins which would bend, and because of previous experience with wrought-iron pins, it was not thought worth while to spend time in testing pins of that material. However, steel pins were tested, including soft and hard steels. The highest carbon content was approximately 1.40 per cent.

None of the steel pins tested in their natural condition gave satisfactory results. Those made of steel with even as high as 0.90 to

1.10% carbon (railway spring steel) persisted in bending before breaking, whereas steels with higher carbon content, though they did not bend unduly, were erratic as to the head or load at which they broke. This was especially true of tool steels. Practically as a last resort, the writer tried the process of hardening the steel; that is, the pins were heated and then hardened by plunging into cold water. Even under these conditions satisfactory results were obtained only within comparatively narrow limits. The softer steels, of course, would not harden, and the very high carbon steels became the more erratic in their behavior by reason of such hardening. Round pins were used, because of the uniform resistance of pins of circular cross-section, no matter at what point the load might be applied (thus making it unnecessary, in this regard at least, to exercise care in placing the pins).

The most satisfactory results were obtained by the use of open-hearth steel with a carbon content varying between 0.40 and 0.80% and phosphorus and sulphur contents each not exceeding 0.05 per cent. In the hardening process the pins were heated to a "cherry red" before quenching in cold water. The heated portion extended about 8 in. above and below that point in the pin which when in place would be at the spillway crest. As the result of the tests on hardened specimens with such characteristics, it was concluded that $1\frac{1}{4}$-in. round pins, with carbon content of 0.62 to 0.77%, aiming at 0.72%, and sulphur and phosphorus contents limited as above, would fail at approximately the predetermined point, *viz.*, when the elevation of the head-water was about 4.3 ft. above the main spillway crests, or about 1.3 ft. above the top of 3-ft. flash-boards. It was also concluded that such pins would not be subject to excessive temporary or permanent deflections before breaking.

As it was impracticable to wait for further tests before making the final selection of the supporting pins to be used, two sets were ordered for each spillway, together with a sufficient number of extra pins to allow for making confirmatory tests. Subsequent analysis showed that the pins delivered actually have a carbon content of 0.68 per cent. The flash-board pin sockets in both old and new spillways were adjusted by grout filling to a uniform depth of 18 in. The pins were cut to 4 ft. 6-in. lengths, so as to allow them to project 3 ft. vertically above

the spillway crests. Seasoned spruce planks were used as flash-boards, with an actual net height of 2.78 ft. above the spillway crests.

The final tests, made on pins chosen at random from those purchased for actual service, showed exceedingly satisfactory results. In no cases where the conditions were equivalent to those of actual service did the pins fail at heads of less than 4.0 ft., nor did they longer than momentarily withstand without failure heads of more than 4.6 ft. above the base of the flash-boards. Moreover, the maximum temporary deflection above the top of the sockets, with the head-water at elevations slightly less than that at which failure occurred, was only about 10° down stream from the vertical. In those instances where the head-water was allowed to subside to zero after having very nearly, but not actually, attained the predetermined elevation, the pins showed a permanent deflection or set of less than 3° from the vertical. The results of these tests, made in the outlet channel, showed surprisingly little difference between the effects of turbulent and of still head-water.

The apparent brittleness of the hardened pins is the more remarkable in view of the "qualifying" test to which the pins had been submitted after they had been hardened, *viz.*, they had been dropped from the crest of the dam to the concrete flooring of the outlet channel, a distance of more than 50 ft. In order to make doubly certain that defective bars were eliminated, each pin was dropped twice in this manner. Sometimes they fell across previously dropped pins, thus concentrating the impact at a single point of the pin. Yet very few which were put through this conclusive test were thereby broken. This "qualifying" test was applied, not merely to the pins used for the confirmatory tests in the outlet channel, but to all pins which were placed in actual service.

With regard to the sensitiveness of the pins in question, the results of an earlier test may be of interest. In that test $1\frac{3}{16}$-in. round pins, with carbon content of 0.62%, were used at the regular intervals of 3 ft. 6 in. to support flash-boards about 38 in. high. The head-water was maintained for $10\frac{1}{2}$ hours at an elevation approximately 3.7 ft. above the base of the flash-boards. At the end of that period the head was increased to 4.2 ft., and within a few minutes two of the three pins under test, supporting three panels of flash-boards, snapped off simultaneously at the top of the sockets.

Since the reconstruction of the dam has been completed, the headwater, with the flash-boards in place, has reached Elevation 139.83, which is about 12⅝ in. over the top, and 3.83 ft. above the base, of the flash-boards. The supporting pins met the expectations, as none failed, despite the fact that they were under severe load for several hours. Subsequently, the flash-boards were taken down for the winter, and it was found that the pins had suffered no permanent deflection.

Underpinning Problems at the New Spillway.—The original footings of Buttress 10, which was left intact and forms the west abutment of the new spillway, were about 22 ft. higher than the footings of the adjacent new Buttress 11, and about 27 ft. higher than the footings of new Buttress 12. The footings of the latter buttresses rest on bed-rock, whereas the original footings of Buttress 10 rested on the clayey over-burden. Evidently, it was necessary to provide secure support for Buttress 10 as well as to insure the stability of the foundation soil under the adjacent (up-hill) portion of the original structure. This was accomplished by underpinning Buttress 10 with a retaining wall founded on bed-rock. In this manner it was possible, in effect, to extend the footings of Buttress 10 down to rock and, at the same time, prevent the lateral displacement of the soil under the adjacent footings. The retaining wall was extended down stream as a portion of the new spillway channel, and decreases in height as the channel swings away from the west bank. It is of reinforced concrete, and is of substantial proportions for a sufficient distance down stream from the footings of Buttress 9 to confine adequately the foundation soil immediately down stream from that portion of the dam, as shown by Figs. 16 and 17.

The type of construction of the underpinning at Buttress 10 is illustrated in Figs. 25 and 26, which show the underpinning in successive stages of construction. To avoid the necessity of temporary shoring under Buttress 10, relatively narrow excavations were first made for the 4-ft. wide counterforts, leaving the intermediate soil to take the weight of the buttress and its overhanging footing (which was not removed). Due to the character of the soil in question, it was impracticable to allow the buttress to be supported in this manner for any considerable length of time. Fortunately, no severe rains occurred in the meantime, and the counterforts were constructed without mishap up to the bottom of the footings. Then, by means of holes

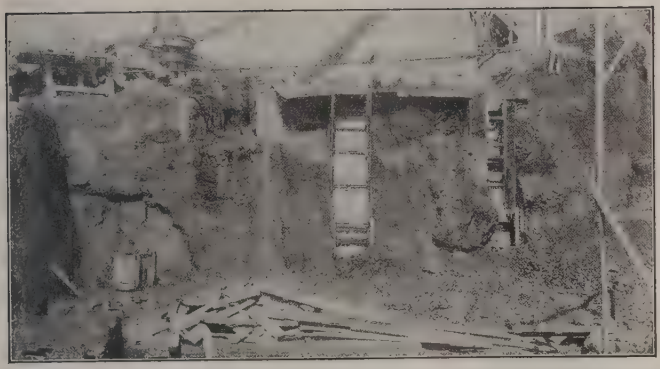
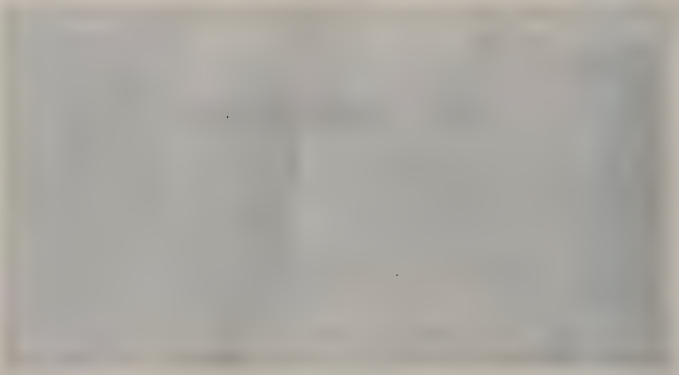


FIG. 25.—UNDERPINNING AT BUTTRESS 10.

FIG. 26.—UNDERPINNING AT BUTTRESS 10, STONY RIVER DAM.



drilled through the footings, grout was forced between the surfaces of contact. Subsequently, the counterforts took the load without noticeable sinking on the part of Buttress 10.

As soon as the concrete in the counterforts had set sufficiently, a reinforced concrete panel, 4 ft. thick, was constructed, bearing against shoulders which had been left in the counterforts. The concrete in this panel was poured directly against the clayey bank, so as to prevent the formation of voids and consequent settlement. All voids left immediately under the footings were filled with grout, not so much to enable the west half of the Buttress 10 footings to transmit load into the foundation soil as to prevent lateral displacement of the soil underlying Buttress 9 and its footings. The retaining wall panels were provided with weep-holes in order to drain adequately the supported soil.

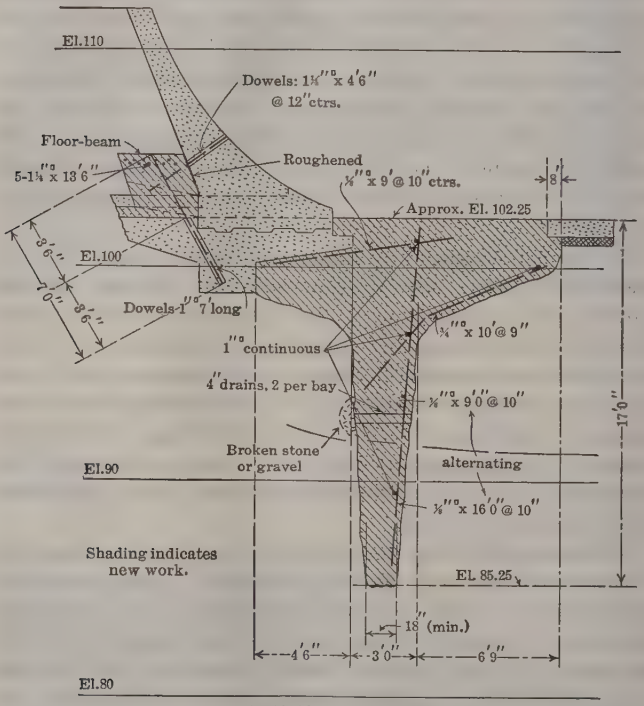
In view of the fact that Buttress 10 now is founded on rock, whereas the adjacent westward Buttress 9 rests on clayey soil, it is probable that the footings of the latter will deflect more than those of the former in transmitting vertical load. Such inequality in deflection would cause undesirable stresses in the footings of Bay 10 were it not for a vertical seam which was cut entirely through the flooring, along the center line of the bay. Thus the footings of the adjacent buttresses can act independently as far up stream as the cut-off wall. The deck of Bay 10 will presumably adjust itself to any such unequal deflection without ill effects.

At the east side of the new spillway a like problem was encountered in connection with the footings of Buttress 19. It was treated in a similar manner, though an additional counterfort was constructed. Here, too, a retaining wall was extended down stream to form the east wall of the new spillway channel, supporting the west flank of the fill which was made immediately down stream from the dam between the old and new spillways.

Toe Protection at Old Spillway.—The channel down stream from the old spillway has a concrete mat 9 in. thick, the mat having been placed on crushed stone which covered the original ground surface to a depth of 4 in. In the reconstruction it was felt that, considering the possible effects of frost, in the way of heaving the channel mat, and of scour at the down-stream end of the mat, such

construction could hardly be considered permanent, and it appeared to be wise to make more substantial provision.

This took the form of additional toe protection, rather than of doing away with the original channel mat and substituting a relatively expensive, heavier mat. The principal features of such toe protection are shown in Fig. 27. The row of 10 by 10-ft. mat slabs immediately down



TOE-WALL AT OLD SPILLWAY
FIG. 27.

stream from the old spillway apron bucket was taken out, and excavation was made for a toe-wall extending to a depth of 17 ft. below the top of the channel mat or hearth. This toe-wall was designed to act as an anchoring wall, similar in this respect to the anchoring walls provided for the bulkhead portions of the original structure. However, it was carried to a greater depth, in order to prevent the undermining

of the main footings of the old spillway in case the channel mat should be washed away in whole or in part.

Here, again, the toe-wall forms, in effect, an extension of the original footings. Its load is received through the original toe of the spillway. However, because the apron of the spillway had been cast separately from the footings and the buttresses, it was impossible to utilize the apron, as originally constructed, to transmit any load into the toe-wall, except in so far as the weight of the apron and the frictional resistance, along the surfaces of contact between the apron and buttresses, were concerned. It was found advisable to utilize the apron to a greater extent by constructing a so-called floor-beam, as shown in Fig. 27, and tying the apron into the floor-beam with the steel dowels there shown. This arrangement also served the purpose of distributing the load on the base of the buttress over a greater distance (up and down stream), thus reducing the unit load on the concrete in proportion.

In addition to the deep toe-wall, wing-walls were extended down stream, as shown in plan on Plate XI. The inner end of each wing-wall extends to the same depth as the toe-wall, but the outer ends are approximately 11 ft. deep below the top of the mat. The wing-wall at the west end of the old spillway was capped by the concrete of a wasteway, which allows any excessive head of water in the channel immediately down stream from the bays of maximum height to relieve itself into the old spillway channel. The new fill between spillways has been protected on its east flank along the old spillway mat by the construction of a wall of sandstone boulders laid in cement mortar.

Anchoring Wall at Heel of Gate Sections.—At Bays 30 and 31 the openings from the reservoir to the outlet gates prevented the construction of the standard type of "heel" shown on Plate XII. The upper cantilever member of the anchoring wall would have interfered with the opening. Therefore, a modified method of construction was used, as shown on Plate XIII. In this case the anchoring wall extends from the top of the concrete protection mat, immediately up stream from the gate openings, to a depth of about 15 ft. below the original footings. The cantilever arms are not so long as in the standard "heel", for the reason that the tie-steel enters the anchoring wall at the middle, inclining downward from the original footings. The steel bars find embedment in the new concrete immediately adjacent

to, and bonded into, the lower portions of Buttresses 29, 30, and 31, as shown in Section *B-B* of Plate XIII. The bars were encased entirely in concrete to protect them against corrosion in the foundation soil.

In order to place the tie-steel, holes were opened by blasting through the original cut-off wall, and tunnels were carried upward under the footings to connect with openings which had been cut down through the footings. The bars were then placed and the concrete poured from the interior of Bays 30 and 31. The special construction at these bays was simpler, in so far as the anchoring wall itself was concerned, but was more expensive than the standard type because of the necessity of placing the tie-steel in the manner described.

Outlet Channel.—The outlet channel has been referred to and partly described in preceding portions of the paper. Its purpose is essentially to prevent any scour which might result either from discharge from the sluice-gates or from water flowing over the intermediate spillway which extends the length of the bulkhead section between the old and new main spillways. The sides of the channel are constructed as retaining walls to hold the down-stream fill. Longitudinal and transverse cross-sections of the outlet channel are shown in Fig. 24, and Plate XI shows it in plan. The channel extends a total distance of 64 ft. down stream from the toe of the original structure, and is protected by a cut-off, about 12 ft. deep, penetrating a bed of heavy boulders. The details of this structure are not such as to warrant further description.

A connecting channel was necessarily constructed to lead from the outlet channel directly down stream to the old river-bed. This change of stream channel has a further advantage in that it protects the lower end of the old spillway channel mat from the scour, due to sluice-gate discharge, to which it was subjected under the original construction.

Materials of Construction.

Cement.—The cement for the work of reconstruction, as also that for the original construction, was furnished by the Alpha Portland Cement Company, and all the cement for the reconstruction came from its Lehigh Valley mills. The cement was required to meet the Specifications of the American Society for Testing Materials plus those specifications of the United States Government (Bureau of Standards)

pertaining to insoluble residue and loss on ignition. It was tested at the mills by representatives of the Pittsburgh Testing Laboratory.

Stone.—Crushed sandstone was used for the concrete aggregate. It was quarried from ledges exposed along the mountain side within convenient distance of the work. Three quarries were opened; two of these required short hauls on the temporary, standard-gauge railroad, and the third furnished rock to the crushers directly by small cars operating on a narrow-gauge track. The cost of quarrying was heavy, inasmuch as the ledges were in no case of great depth, and the cost of stripping was therefore proportionately high.

In view of the relatively high cost of the materials for concrete, sandstone boulders were utilized to as great an extent as possible. These boulders were ordinarily of one-man size, and either were found in the river-bed or were yielded by the excavation for anchoring walls, cut-off extensions, etc. The larger proportion of the boulders was found in the excavations near the middle of the valley.

Sand.—As in the case of the original construction, sand was obtained by crushing and rolling the native sandstone, though the product was not the best that could be desired. Probably the chief defects in the sand were its inferior grading, and especially the large proportion of fines. Nevertheless, the resulting concrete was acceptable, as shown by tests made from time to time on standard specimens cast in most cases from concrete taken directly from the mixer in the regular course of the work. The specimens were shipped to Pittsburgh to be tested.

Steel.—Open-hearth reinforcing steel was furnished by the Carnegie Steel Company, under the June 1st, 1912, Standard Specifications, of the American Society for Testing Materials, for cold-twisted, steel reinforcing bars. Thus the steel was required to show a minimum yield point of 55 000 lb. per sq. in. The standard design data, as used for the cold-twisted bars, may be of general interest, and are therefore submitted in Table 6.

Classes of Concrete Construction.—Four standard classes of concrete construction were used in the reconstruction, and, on the basis of these classes, working stresses for use in design were specified, as follows:

Class A-1.—Reinforced concrete; proportions 1 : 2¼ : 3¾; allowable working stress for steel, 18 000 lb. per sq. in.; for concrete,

TABLE 6.—Standard Design Data for Cold-Twisted Square Steel Bars.

Diameter or size of bar, in inches. d	*Minimum embedment, surface of concrete to center of bar, in inches.	†Lap or Length of Embedment Required to Develop Maximum Safe Unit Stress in Bar, in inches—L.				$10d$ Radius for bend, in inches.	†Lengths—see Diagram.		‡Allowable range of spacing between centers of bars, in inches.	Preferable spacing between centers of bars, in inches.
		At 16 000 lb. per sq. in.			At 18 000 lb. per sq. in.		a	b		
		Bond stress, in pounds per square inch.					In inches.	In inches.		
		75 (A-2)	67 (B)	60 (C)	75 (A-1)					
⅜	1½	20	23	25	23	4	12	8	1½ to 6	2
½	2	27	30	34	30	5	16	11	2 to 6	2½
⅝	2½	33	38	42	38	6	19	13	2½ to 10	3
¾	3	40	45	50	45	7½	24	16½	3 to 12	4
⅞	3½	47	53	58	53	9	28	19	3½ to 13	4½
1	4	54	60	67	60	10	32	22	4 to 14	5
1¼	4	67	75	84	75	12½	40	27½	4½ to 16	6

Note.—Use no ⅜-in. bars except under especially approved conditions.

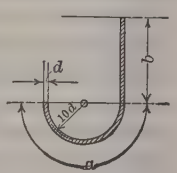
Use ½-in. bars only where unavoidable, *e. g.*, for slabs only 6 or 8 in. thick.

As far as possible use bars of same diameter in same portion of structure, so to avoid confusion.

Wherever possible, bar lengths are to be specified to nearest ½ ft.

Bars should not be specified more than 40 ft. long; and ordinarily not more than 30 ft. long.

Use no clamps for splicing unless there is no other recourse.



*These dimensions apply likewise to distance from end of bar to nearest concrete surface. Where bottom of slab is cast on ground, allow 1 or 2 in. extra depth of embedment.

†Where available length for straight embedment is less than 90% of standard length, use 180° bend. (See diagram.) Letters in parentheses refer to classes of concrete.

‡Bars, in general, should not be farther from center to center than the distance from outer compression fiber of concrete to center of steel.

700 lb. per sq. in. This was used almost solely in the footing strengthening, because the protected condition of the concrete there warranted taking advantage of the economy resulting from the highest practicable working stresses.

Class A-2.—Reinforced concrete; proportions $1 : 2\frac{1}{4} : 3\frac{3}{4}$ (the same as those for the preceding class); allowable working stress for steel, 16 000 lb. per sq. in.; for concrete, 500 lb. per sq. in. This class of construction was used for the deck, apron, and brace-beams of the new spillway; also for all reinforced concrete retaining walls.

Class B.—Reinforced concrete; proportions $1 : 2\frac{3}{4} : 4\frac{3}{4}$. This class of construction was used where the design was not limited to the bare requirements measured by allowable working stresses,

and where the quantities of concrete were ample. It was used, for instance, in the buttresses of the new spillway, the anchoring walls at heel and toe of the original structure (excepting the toe-wall at the old spillway), curtain-walls and roofs, main parapet of the outside bulkhead sections, floor of the outlet channel, and the heavy mat of the new spillway channel.

Class C.—Plain concrete; proportions 1 : 3½ : 5½. In general, this class of construction had no reinforcement, except for temperature stresses and the like. It was used for the buttress footings and the combination anchoring wall and cut-off of the new spillway, the counterforted retaining walls underpinning Buttresses 10 and 19, all cut-off wall extensions and the toe-wall at the old spillway. In the latter case, especially, as well as in certain other places, considerable quantities of boulders were incorporated in the Class *C* concrete. It was specified that boulders should not constitute more than 40% of the total volume, and that they should not interfere, or be in contact, with reinforcing steel.

Construction Methods and Equipment.

The total quantity of concrete involved in the reconstruction and strengthening work was approximately 10 160 cu. yd., according to the plans; actually, about 11 700 cu. yd. (including boulders) were placed, the discrepancy of about 15% being due principally to over-run in trench excavation. The quantity of concrete reported to have been placed during the original construction of the dam was 11 800 cu. yd. Evidently, the reconstruction was by no means a small matter as compared with the original construction. The quantities involved in each case were less than those which would have been required had a dam of the solid masonry type been constructed; yet the complexities involved in, and the attention to detail required by, the type of construction actually used were very much greater than would have been necessary in the case of a solid dam.

The construction methods were essentially like those used in the original construction work, and described elsewhere.* In view of that fact, it does not seem proper to occupy space in this paper with any detailed description of the construction methods. As a matter of fact,

* *Engineering News*, January 22d, 1914.

certain special methods of procedure applied in the reconstruction have already been described in connection with the discussion of the governing features of design.

Construction Equipment.—As soon as practicable after the writer had been engaged to investigate the foundation conditions and determine the feasibility of reconstructing the dam so as to be safe, work was begun on a standard-gauge railroad connection. This was made, as in the case of the original construction, with the standard-gauge logging railroad of the Parsons Pulp and Paper Company, but at a point nearer the dam site and about 16 miles from Dobbin, W. Va., on the Western Maryland Railroad. The connecting spur added a little more than 2 miles to the distance. It was constructed at a cost of about $3 000 per mile for grading the road-bed, ballasting, cross-ties in place, and laying rails, though not including the cost of the rails.

This low cost was due to the fact that, in the construction of this spur, no pretense was made of keeping curvature and grades within ordinary limits, nor was much attention paid to ballasting the road-bed. The maximum curvature was 31° 15′, and the maximum grade was 12.4 per cent. For a considerable distance the grade averaged 9.8 per cent. The contour of the ground was followed closely, and the greatest part of the work of grading consisted in taking out boulders with which the ground was strewn. Small streams were bridged with tree-trunk stringers or cribbing.

The rails were new, 85 lb. per yd., and were carried on heavy ties, which, however, were spaced at greater than standard distances. After the reconstruction was completed, the rails were taken up and transferred for permanent use by the owner in lumbering operations.

Despite the inferior character of the roadbed, no accidents worthy of note occurred, and the results were such as to justify what might have been an inadequate expenditure for railroad connection, had it not been for the relatively heavy rails. The owner supplied a 42-ton geared engine from its West Virginia lumbering operations, and three of its own standard 40-ft. flat cars which were used principally for transporting rock from the quarries to the crushers. The engine was able to handle one standard loaded box car at a time up the maximum grade. Fortunately, the grade favored in-bound freight.

A Marsh-Capron mixer, of ¾ cu. yd. capacity, steam-engine driven, mixed the concrete and delivered it into buckets of 1½ cu. yd. capacity.

set on flat cars. These cars were run on a narrow-gauge track from the mixer out to a point near the west abutment of the dam and directly under a 3-ton Lidgerwood cableway. The cableway was of 1 000 ft. span, and was used in turn to deliver the buckets of concrete either to hoppers and chutes for distribution laterally and under the deck of the original structure, or, in the case of the new spillway buttresses, into wooden bucket cars which ran on portable, narrow-gauge tracks along the top of the buttress forms, and thus allowed the concrete to be deposited near the ends of the forms. The cableway was also of general utility in shifting equipment and forms, transporting reinforcing steel, etc.

Steam was furnished both from a main plant, consisting of two 100-h.p. boilers, on the west hillside just above the spur railroad track, and also from smaller, individual boilers placed at different parts of the work from time to time, according to immediate needs. Such boilers were especially necessary during cold weather because of the condensation in the long pipe line from the main boiler plant. The walkway through the original structure, and continuing through the new spillway, served as a convenient carrier, on which were placed water, steam, and compressed air pipes, which were tapped wherever necessary. Water was supplied from a duplicate set of steam plunger pumps, which were set up in Bay 30 and took water from a sump fed from the river by the 20-in. sluice-gates.

An 8 by 10-in. compressor proved to be a great utility, especially in connection with the very convenient Ingersoll-Rand "jack-hammer" drills. The latter were used to drill holes in the original concrete, for the purpose of embedding steel dowels, or for blasting—as was especially necessary in cutting out the base of the deck to allow the passage of the tie-steel from the anchoring wall at the heel. These air drills were also used for roughening the original concrete wherever necessary. Operated by steam, they proved only fairly satisfactory for drilling purposes in the stone quarries, but were displaced later by a standard Sullivan steam drill.

For pumping, an 8-in. centrifugal pump and a portable, gasoline-engine driven, 4-in. diaphragm pump, were used as much as possible, but in the deeper trenches steam siphons were used.

Cold-Weather Concreting Precautions.—Concrete was placed throughout the winter of 1914-15 under temperatures as low as 10°

Fahr. The principal precautions to prevent concrete from freezing were: First, the water used in mixing the concrete was heated so that the temperature of the mixed batch, after being away from the mixer several minutes, was about 55° to 75° Fahr. The water was heated by live steam to a temperature considerably higher than that last mentioned because the aggregate absorbed much heat from the mortar of the batch, despite the fact that steam pipes were used in the sand and crushed stone supply bins to heat these materials also.

Secondly, the concrete or earth surfaces against which new concrete was to be poured were thawed out or warmed. This was accomplished with a jet of live steam which at the same time served to clean thoroughly the surfaces of old concrete.

Thirdly, the freshly poured concrete was covered with canvas. The canvas covering was usually arranged so as to leave an air space which, under especially severe temperatures, or with thin concrete, was warmed with coal-burning salamanders or gasoline torches. At times, also, live steam was turned in under the covering until the concrete had attained a sufficient set.

As the result of such precautions, little concrete was frozen. Those surfaces which were frost-bitten were usually in places where the concrete was relatively massive, and precautions had consequently been relaxed.

Cost.—Despite the fact that approximately the same quantity of concrete was placed in the reconstruction and strengthening as in the original construction, the later work was done at a considerably less total cost, due principally to the fact that much of the new concrete was comparatively massive. Nevertheless, the new work required more attention and care than if the work had been entirely of a standard nature and free from the conditions imposed by the existence of the original structure. Only about 25% of the total cost of the reconstruction and strengthening was expended in rebuilding (as a new spillway) the portion of the dam between Buttresses 10 and 19. Had this portion been rebuilt as a bulkhead section, rather than as a new spillway, the proportion of the total cost involved therein would have been even less. The remainder of the cost was expended in strengthening the original structure and in providing features which had not at first been supplied. A considerable quantity of work, especially in the form of

cut-off wall underpinning, was not anticipated at the beginning of the reconstruction.

Due to the facts that the new work was scattered over the entire site and that it was carried on through the winter, the unit costs were necessarily high. The fact that the work did not conform to standards (except in the case of the new spillway superstructure, which constituted a small proportion of the whole) also contributed to such high unit costs. For the same reason, the unit costs are not of general value. Thus the monthly average costs for excavation, exclusive of charges for plant and overhead expenses, varied from \$0.51 for new spillway channel excavation to \$7.68 per cu. yd. for excavation required for underpinning the original cut-off near the center of the valley. The corresponding costs for concrete varied from \$3.40 per cu. yd. in place for Class *C* concrete in cut-off extensions to \$13.43 per cu. yd. for Class *B* concrete in curtain-wall and roofs.

The results attained in the reconstruction and strengthening of the Stony River Dam could have been secured at far less cost—undoubtedly less than half the cost of reconstruction—had the features involved been incorporated in the original construction.

Personnel.

In his capacity as Consulting Engineer in charge, the writer made inspection trips to the work about twice each month, after the reconstruction had been begun in earnest. Mr. D. N. Showalter represented the writer as Resident Engineer throughout the entire reconstruction. Mr. C. W. Hotaling was Superintendent of Construction, having filled the same position for the Ambursen Hydraulic Construction Company after it took over the original construction work in March, 1913.

The writer desires to take this opportunity to express his appreciation of the whole-hearted co-operation of Messrs. Showalter and Hotaling, as well as of the executive officers of the West Virginia Pulp and Paper Company, owner; also to acknowledge the valued advice of Daniel W. Mead and C. V. Seastone, Members, Am. Soc. C. E., with whom he was privileged to consult on several occasions during the period of reconstruction.

DISCUSSION

Mr. Ledoux.

J. W. LEDOUX,* M. AM. SOC. C. E. (by letter).—Mr. Scheidenhelm has written a very exhaustive treatise on the reconstruction of the Stony River Dam. As a result of his work he has formed two important conclusions with which the writer heartily agrees and has been advocating for a great many years, namely, that most masonry dams fail due to sliding, and that too little provision is made for excessive floods.

Heretofore, the engineer was considered conservative when, on a small water-shed, he provided for a flood of 250 ft. per sec. per sq. mile, but floods of several times that size are likely to occur, and it is only a question of time before they do.

Whether provision should be made for the greatest possible flood that will probably occur at intervals of a calculated number of years involves several questions:

First.—Complete provision should be made, if otherwise lives may be lost.

Second.—If only damage to property and the structure itself is involved, then it becomes a question of whether the cost of making complete provision is greater or less than that of the damage when figured on a present-worth basis. For instance, if a dam is seriously overtopped once every 50 years, causing a wash-out which is estimated to cost $20 000 to repair, and the cost of making complete provision to prevent that overtopping would be $10 000, the present worth of $20 000 to be spent 25 years later at 5% is about $6 000; so, on this basis, it would be engineering prudence to take a chance, provided no other inconvenience or damage were entailed than the $20 000. It involves great expense in many cases to make provision to take care of a flood equal to 1 600 sec-ft. per sq. mile. Small water-sheds in the eastern part of the United States, however, are subject to such floods at very rare intervals, and that means a flow corresponding to a continuous rainfall of about 2½ in. per hour.

The writer visited the Stony River Dam a few days after its failure. The following is an extract from his report made at that time:

"The caretaker, Mr. Kerr, who lives with his wife at the house close to the dam, stated that on Wednesday morning, January 14th, about nine o'clock, he noticed water coming through muddy on the down-stream side of the dam at the point where it subsequently failed. By Wednesday night this had become much worse, and on Friday morning at 1 o'clock he notified Mr. Allen Luke, one of the members of the firm of the West Virginia Pulp and Paper Company, that he was afraid the dam was going to break. He was told to open the gates located on the inside of the dam. He stated to his people that these

* Philadelphia, Pa.

gates could not be opened on account of the stairway leading to them being covered with ice, the temperature being below zero. Mr. Ledoux.

"By 2.30 A. M. conditions had become so grave that he notified by telephone the town of Maysville and others located down stream along the banks of the Stony River and Potomac. The water was rushing through the large and increasing space underneath the dam, and by Thursday morning at 9 o'clock the water had fallen in the dam 3 ft. At 9.30, two bays, or 30 ft., broke down, and in half an hour three more failed. By 11 o'clock the water was about 13 ft. below the spillway, and by 6 P. M., only 4 or 5 ft. of water remained in the dam. By Friday morning it was entirely empty.

"The dam was built on a clay formation and under the up-stream toe was a cut-off wall, from 3 to 4 ft. thick. For a distance of 600 ft. or more, on the east side of the valley, this cut-off wall extended down to rock, which, from all accounts, is of an indifferent quality, characteristic of the bituminous coal formation in the vicinity of the Middle Kittanning. The cut-off trench proved to be very difficult, due to its depth, the number of boulders encountered, and water-bearing soil, so that the material sometimes caved behind the shoring and made the work dangerous. The engineers in charge of the work figured that a considerable amount of money could be saved by not continuing on rock, which appeared to be increasingly difficult to reach toward the west end, and so notified the principals of the Paper Company. Not wishing to take any chances, the Company advised a conference with able consulting engineers, which was had, and it was decided to step up the cut-off wall until it had a depth of about 5 ft., and, according to information given me by the Superintendent of the Whitmer Lumber Company, these engineers concurred in the recommendation of the local engineers that it would be safe to make this change, on account of the quality of the material being very compact clay, sand, and gravel.

"At the west end of the break, this cut-off wall was not more than 5 ft. deep and about 2½ ft. in thickness. The break extended from Abutment 11 to Abutment 16, and the dam is further damaged to Abutment 19. The stepping up from rock to earth began suddenly between Abutments 15 and 16, the first step being about 11 ft., and at Abutment 11, the bottom of the cut-off trench was more than 20 ft. higher than it was at Abutment 16. It is quite probable that unequal settlement occurred due to the change from rock to earth, as this is where the break took place, and it is hard to understand how engineers familiar with the Austin Dam failure could have consented to any such curtailment of depth of the cut-off trench.

"The writer does not think that the failure of the dam was due to the Ambursen type of superstructure, but it is very clear that, with this type, it is no more safe to economize on the depth of foundation than with any other, because if it is built on an earth foundation and water can flow through under it in material quantities, failure is certain to take place.

"The strongest claim of the advocates of the hollow dam, particularly the Ambursen, is that it is much more economical than the solid gravity type, but, as a matter of fact, on account of its structural complication, with its forms, reinforcement, and necessity of more expensive concrete, it costs not much less for equal stability and utility;

Mr. Ledoux.

but, in order to make the comparison more favorable to their type, they economize in the foundations, which, in the writer's judgment, should be as thoroughly taken care of as with the gravity dam. The case in point is certainly a flagrant illustration of their bad judgment in this respect.

"According to the plans of the dam revealed in a paper presented by Mr. Bayles in *Engineering News*, of January 22d, 1914, it is going to be a very difficult matter to decide just how far to carry the repairs so as to prevent subsequent failure of some other portion.

"It is the practice of conservative engineers never to build a masonry or concrete gravity type of dam unless they can found it on rock for practically the entire distance, and, in addition thereto, it is the practice to carry a cut-off wall down far enough underneath the main foundation to be practically impervious to water. The writer does not see what advantage the Ambursen type of construction possesses which makes this requirement unnecessary, but, in the case of the Stony River Dam, none of the main structure was founded on solid ledge rock and only two-thirds of the cut-off trench.

"Fortunately, no lives were lost, but that was probably due to the gradual emptying of the dam. If the break had taken place suddenly, when the dam was full, like that at Austin, Pa., it is probable that there would have been some loss of life, although the towns were far enough down stream so that they could be notified by telephone at least an hour before the water reached them.

"From an inspection of the construction, which is very readily made from a concrete walk, running longitudinally within and throughout the entire length of the dam, the writer is convinced that the superstructure was built in a workmanlike manner, but the concrete appears to be somewhat softer than it should, according to the best practice. On account of the large amount of snow all around and the débris in the valley on the down-stream side of the dam, and the ice on the up-stream side, it was impossible to make any close inspection of the soil or the geological formations.

"In one of the bays are several gates for letting off the water in case of emergency, or for any other reason, and there is also in an adjacent bay a large gate for the same purpose. An archway entrance goes through the wall between these two bays. At the side of one of them is a concrete stairway and an iron hand railing, and this, at the time, was covered with ice, due to drippings from a crack in the up-stream portion of the dam, making the valves inaccessible. A casual inspection, however, showed that this ice could have been chopped off with a hatchet or axe in a few minutes, or poles or a ladder could have been run down from the concrete walk to the floor where the valves were located in the adjacent bay. If these gates had been opened wide from Wednesday morning, it is quite probable that the water in the dam would have been so lowered as to prevent failure. It is better, however, for the engineering profession, that the dam did fail, as it will further impress upon their minds the necessity of care and thoroughness in the design and construction of dams in general.

"The nature and abundance of suitable soil and the depth of the bed-rock indicate that the proper type of structure would have been an earthen dam with a concrete spillway on either the east or the west

side-hill, and this could probably have been built safely at as low a cost as the type that was adopted." Mr. Ledoux.

J. K. Finch,* Assoc. M. Am. Soc. C. E. (by letter).—When the failure of the Stony River Dam was first reported, and before complete details were available, the writer pointed out† that dams of this type were particularly weak as regards sliding. Articles on the inclined slab and buttress dam always emphasize the great stability of these dams against overturning, but fail absolutely to call attention to their weakness as regards sliding. When the buttresses extend down to rock, the rock surface should be roughened and the buttress well bonded to it. When the dam is founded on a spread footing, four important points must be considered. Mr. Finch.

1.—The material just below the floor of the valley, on which the spread footing between the buttresses is placed, must be of a satisfactory, compact, substantial character that will not "flow" even when wet.

The idea has seemed to be prevalent in the minds of some designers that the slab and buttress dam of this type can be set down anywhere, on the surface of almost any kind of ground. In a pamphlet advertising a dam of this type, it is described as built on "a foundation of quicksand and hay." Without knowing the details, it is difficult to criticize the design, but the writer hardly considers it as conservative engineering. This particular type is advantageous, of course, from the standpoint of economy, when the depth to rock is great, but the foundation material is certain to be wet at times, and a soft rock, hardpan, compact gravel and, in certain cases, a compact clay, would seem to be the foundation range for a dam of this character.

2.—The unit pressure on the bottom of the spread footing should be low and uniform, and the possibility of upward pressure on the footing, or of back-wash undermining it, should be guarded against.

If the slab is inclined at the usual angle of about 45°, uniform pressure at the base of the buttress can be secured by making the base about 1.3 of its height. It is usually assumed that the entire weight of, and pressure on, the slab is transmitted to the foundation through the buttress. A small "wash-wall" at the down-stream edge of the dam prevents undermining of the footing and also confines the foundation material, but, in the writer's opinion, this should not be counted on to resist sliding. Weep-holes are provided in the footing in order to eliminate upward pressure on the base.

3.—The cut-off wall should extend down to impermeable material, below any seams which might communicate between the reservoir floor and the stream bed below the dam.

Practically all the failures of these dams have been due primarily to inadequate cut-off walls. Note especially the Pittsfield, Mass., Dam

* New York City.

† In a letter to *Engineering News*, January 22d, 1914, p. 202.

Mr. Finch.

and the less fortunate Stony River Dam. It is said that engineers should profit by the expensive experience of others, but, unfortunately, some engineers are still willing to "take a chance", usually, however, at the expense of others. It is also far from rare to find cut-off walls built without reinforcement in soft material requiring sheeting and where the sheeting is subsequently pulled. It would assuredly seem that all cut-off walls which are not built in compact earth in an unlined trench, or in a trench in which the sheeting is left in place, should be reinforced.

4.—Stability against sliding must be adequately provided for.

In the letter previously mentioned the writer said:

"A simple calculation shows that in the case of the Stony River Dam a coefficient of friction of about 0.52 is required to prevent sliding. * * * the soil at the dam site [is described] as 'a yellow clay mixed with fine sand or gravel which is underlain by a stiff blue clay, and some seams of a black material occurring in places.' At the best, this material could not be counted on to give a higher coefficient than 0.70 to 0.80 and when wet might give as low as 0.35. Moreover, the design shows weep-holes in the spread flooring, indicating that the designer expected water might, as it doubtless would, occur under the base. Under these conditions it is difficult to see what factor of safety there was against sliding except the shearing value of the concrete of the cut-off wall (which in this design is unreinforced) and the small 4-ft. wash wall at the lower edge of the floor that should most certainly not be counted on as offering any resistance."

Lack of stability against sliding is a well-recognized point of weakness in reinforced concrete structures of the retaining type, and must always receive careful consideration. In retaining walls, it is provided for by projections on the base, and, in the case of a dam of this type, Mr. Scheidenhelm has described the various methods available, and has finally selected the method of utilizing the upper portion of the cut-off wall for this purpose. The writer has been advocating the same method of design in teaching a class in reinforced concrete, and is pleased to see this feature treated so completely and thoroughly in this paper. The slab and buttress dam, properly and conservatively designed, offers economic advantages which undoubtedly make it a most desirable type in certain locations; Mr. Scheidenhelm's paper clears up one of the important points regarding its design, which has long been neglected, and is a most valuable contribution to the literature of the subject. It is indeed unfortunate that the West Virginia Pulp and Paper Company, which, like many other concerns, did not try to economize at the expense of safety, but, on the other hand, insisted from the first on a reliable structure, should have been, through no fault of its own, the victim of such an unnecessary and expensive experience.

Mr. Finch.

One other point in the design of these dams remains to be settled, namely, their architectural treatment. A design for a dam at Coatesville, Pa.,* by Alexander Potter, Assoc. M. Am. Soc. C. E., illustrates the only attempt, so far as the writer is aware, to design one of these structures of really pleasing appearance. In the Stony River Dam, as the author pointed out, no such treatment was considered necessary, but it is more and more evident that, in engineering structures which are near cities or much traveled highways, the appearance must be considered. Too often work of this kind is attempted by a designer who knows nothing of the fundamental principles involved, or it is left to a young architect who is familiar only with house architecture and attempts to apply it to a totally different structure, or applies some inappropriate form of ornamentation that detracts from, rather than enhances, its appearance.

The Engineering Profession lacks in any "tradition", regarding the proper treatment of engineering structures, such as the architect has in the treatment of buildings and on which he is prone to lean, as many believe, too heavily. Many of the engineering works in Europe, which have been held up as examples of good engineering architecture, are merely house architecture applied to engineering work. The true solution of the problem is in developing a suitable treatment, forming, for engineering structures, a tradition based on the same fundamental elements as those on which architecture (building composition) is based, but emphasizing the structural features of the engineering design to a much greater extent than is common in architecture.

The fundamental considerations of unity of effect, balance, proportion, harmony, and climax cannot be taken care of after the structure has been planned and partly designed. They must be kept in mind from the very first and receive their proper consideration in the preliminary schemes, sketches, and plans for the work. The only man who can do this is the engineer himself, who realizes the structural and economic principles involved, and, it is hoped, may soon come to realize the necessity for proper architectural composition as a basis for a final and simple decoration in which the structural features should be emphasized.

Mr. Rutenberg.

P. P. Rutenberg,† Esq. (by letter).—European engineers have learned much from their American colleagues, and the war proves the extent to which American methods of solving technical problems, of their organization and realization, have been adopted and developed in Europe. In the special field of dam building they have heard and learned from such world-famous works as the Croton, the Roosevelt, the Elephant Butte Dams, and many others.

* *Engineering News*, February 26th, 1914, p. 461.
† Petrograd, Russia.

Mr. Rutenberg.

If the writer now takes the liberty of criticizing the construction of the Stony River Dam, it is because this structure, in his judgment, is lacking in just that efficiency which is characteristic of American engineering.

Foundations.—As described by the author, the foundation soil at the site of the dam is mainly of a clayey nature, heterogeneous, pervious, and of poor bearing value in general. The examination of the old cut-off wall, and of the buttress footings remaining intact, showed that abundant percolation and leakage existed in the foundation soil of the original dam. "The contact between the original footings and the foundation soil was in some places found to be faulty. * * * Hollow spaces were found under the footings around the 4 by 12-in. weep-holes." In some bays, it was discovered that "the foundation soil had been washed away at the junction between the footings and the cut-off wall. * * * The construction joint would naturally open somewhat, due to the deflection of the foundation soil when loaded. * * * Observations showed that grout under pressure traveled under the footings for a distance of at least 100 ft.", etc. (pages 987-988).

All this was produced by the leakage under pressure during 65 days only, the period in which the original dam functioned, until it failed.

The faulty conditions of the foundation soil were remedied by pressure grouting through the original weep-holes. In this way, the original foundation soil under the footings was more or less compacted, but there surely remained other hollow spaces and pockets from which water under pressure passed, not to the weep-holes of the footings, but in other directions of least resistance, not observed in the reconstruction of the dam. It will be shown later that the author himself admits it, and that his statement (page 988) that "the foundation soil under the original footings was thoroughly compacted so as to warrant the assumption that the footings are everywhere in contact with the foundation soil", is not entirely correct. Percolation and leakage will cause harmful and dangerous uplift pressure in any dam, with any foundation soil, even the best.

The uplift pressure applied to the up-stream part of the base increases the overturning moment of the dam and the unit stresses in the structure and on the soil at the down-stream foot of the dam, where, under the full load of the dam, the values of these stresses are maximum. The uplift pressure on the up-stream part of the base decreases, consequently, the resistance of the dam to overturning, and may render insufficient the resistance of the structure and that of the soil at the base to the efforts of compression and shearing developing there. The same conditions created by the uplift pressure decrease also the resistance of the dam to sliding. This has been

Mr. Rutenberg

the cause of the failure of many dams, even with good foundation soils which have possessed the best frictional resistance.

Therefore, it is impossible to state, as Mr. Scheidenhelm does (page 947), that "the smaller the coefficient of frictional resistance, the less the net effect of the uplift pressure."

Consequently, it is clear that percolation and leakage must be eliminated by all possible means in any dam, in general, and in the Stony River Dam, with its heterogeneous and pervious clay foundations, in particular.

The interception of the leakage and percolation in this dam should have been made directly and immediately on the up-stream face of its cut-off wall and discharged down stream by special channels without any pressure.

Now, the principles on which the Stony River Dam was reconstructed are quite different. On page 997 we read:

"It was believed to be preferable, therefore, to take such risk as might result from the existence of pockets, as it were, of unrelieved uplift pressure near the down-stream side of the cut-off wall rather than to invite further leakage by tapping such relatively harmless, deep-lying pockets of water under pressure."

This statement, it seems to the writer, does not tally with the most vital principles in dam-building, namely, that water under pressure in the foundation soil is never harmless.

According to the foregoing consideration, the system of drainage adopted for the Stony River Dam consists of 3-in. wrought-iron pipes, perforated with four $\frac{3}{8}$-in. holes per linear foot, one hole in each quadrant; three pipes per bay, driven into the foundation soil vertically, the first "about 20 ft. down stream from the cut-off wall, except at the bays of lesser height, where they are at a minimum distance of 12 ft. from it."

The writer believes that:

1.—A 3-in. pipe with four small perforations per linear foot for a bay 15 ft. high is useless for drainage, even if the drainage pipes described by the author function perfectly well.

2.—The small perforations in the pipes must undoubtedly have been clogged with clay in the process of being sunk. Mr. Scheidenhelm states (page 998) that "any considerable hydrostatic pressure near the drain pipes would soon open the perforations." He admits the presence in the foundation soil of considerable hydrostatic pressures at a distance of 20 ft. from the down-stream side of the cut-off wall, which is about the middle of the base. Consequently, by the improper location of the drainage pipes, alone, the up-stream half of the base is exposed to dangerous uplift pressures, with all previously stated consequences, *viz.*, decrease of resistance by overturning, increase of

Mr. Rutenberg.

the unit stresses in the structure and on the foundation soil, saturation of the clay foundation soil with water, and decrease of its resistance to sliding; which, as the writer understands, cannot but be harmful, and all this solely because the drainage pipes, even assuming that otherwise they function perfectly well, are not in the proper places.

3.—Some of the drainage pipes have been driven vertically to a depth of 20 ft. into the foundation soil. When they are filled with drained water, it is clear that their lower perforations are submitted to the pressure of a column about 20 ft. high, and at this lower stratum of the foundation soil the leakage will not be able to penetrate into the drainage pipes, unless it is of greater hydrostatic pressure. Only a small part of the leakage will penetrate through the small perforations; the remainder will continue its way down stream into the foundation soil, where the resistance is weaker. Besides, there is a possibility that the drainage pipes will be filled through the higher perforations from the higher strata of the foundation soil, and that the collected water will be conducted through the pipes and their lower perforations, by the water pressure thus formed, into the otherwise dry lower strata of the foundation soil.

In fact, on page 1000, the author takes cognizance of such phenomena, for which manifestation he "has been unable to advance any explanation which is satisfactory to himself."

The writer, therefore, considers these pipes, not as a drainage system, but rather as a means of introducing water into the foundation soil, and not only useless, but dangerous to the safety of the dam.

Resistance to Sliding.—On page 908, we read:

"The method adopted [for increasing the resistance to sliding] is believed to be new, and consists in the use of anchoring walls extending to a considerable depth into the underlying foundation soil, and, in effect, utilizing the weight of that underlying soil as well as the resistance (to horizontal movement) of the soil immediately down stream from the structure."

These anchoring walls are two: one at the heel, and another at the toe, of the original structure which remained intact.

To render the above-mentioned service, the heel-wall must be strong enough to be able, in case of a down-stream sliding of the dam, to function as a cantilever which can overcome the resistance of the underlying foundation soil to sliding and shearing.

The paper does not give the dimensions and calculations of the anchoring walls, but, from Plate XII, it is clear that they are too weak to serve this purpose. The heel-wall is not able to compact the underlying foundation soil, and, therefore, make it less pervious by any down-stream movement on the part of the dam, as the author believes. It will simply be broken in this case, opening the joint

between the upper structure and the cut-off wall, as happened in the original structure, according to the description on page 959. Mr. Rutenberg.

The toe-wall, as constructed, will aid in collecting the leakage under the dam. Consequently, it will increase the uplift pressures and the quantity of water in the foundation clay, decreasing the resistance of the dam to overturning and sliding.

Openings should have been made in the vertical part of this wall, directly under the footing floor of the dam to provide a free exit for the leakage.

As far as the two anchoring walls are concerned, the writer does not agree with the author's assertion that "the reconstructed dam acts essentially as a monolith" and, consequently, he does not agree that "the horizontal member of the toe-wall in effect increases the width of the base of the dam, thus decreasing the unit vertical load on the foundation soil." (Page 960.) The efforts transmitted by the buttresses to the foundation soil have their maximum value at this point, and, the writer believes, under the existing circumstances, will rather break this horizontal member of the toe-wall.

The under-cut slope of the heel-wall presents, besides, a surface for the application of uplift pressures, which, with their big moment arm, decrease also the stability of the original structure against overturning.

In the reconstruction of the intact part of the dam, the only things to do were: to fill, if possible, all the holes found in the foundation soil; to repair the cut-off wall, and to build a reinforced concrete wall up stream from the old cut-off wall, bearing upon this by concrete apertures, in order to form chambers which would be able to maintain the percolation through this drainage wall and conduct it without pressure down stream from the dam by convenient channels.

The two anchoring walls with the drainage pipes have weakened the original stability of the dam.

The "new theory" of resistance to sliding, with the investigation about the "plane of least resistance", with the tests, calculations, and deductions of an "average" coefficient of frictional resistance of 0.61 in wet clay, is an interesting theoretical exercise, but without practical value for the reconstructed dam. One is the frictional resistance of a cubic foot of clay, pulled by two men under the conditions of the improvised laboratory described; the other is the effective frictional resistance of the wet clay of heterogeneous structure in the real foundation soil loaded by a head of 90 ft. and more, or by about 6 000 lb. per sq. ft.

There are dams in which the necessary weight is obtained by clay, sand, pebbles, even water, included in a reinforced coffer, the bottom of which constitutes an elastic monolith with other coffers.

Mr. Rutenberg.

If percolation took place, it would be applied to the bottom and, consequently, to the whole structure of the dam. The material of light weight cannot be washed from them.

In the Stony River Dam, however, engineers cannot seriously consider for its resistance to overturning and sliding, the "weight" of the underlying foundation soil, which can be and is, as is known, washed away by the existing leakage and percolation.

Spillway.—Not being acquainted with the hydrological conditions of America, the writer cannot give an opinion as to whether the spillway capacity provided in the reconstructed dam is adequate or excessive; but one thing is clear: that by adding 3½ ft. of water to the original level in the reservoir, the horizontal water pressure and its overturning moment are increased by about 20% for the typical section indicated on Plate XII. The increase is greater for the sections of less height.

The parapet-like addition to the original structure and the curtain-wall and roof, contribute a vertical effort of important value, which passes into the down-stream third of the base. Both conditions increase the overturning moment, consequently, they also decrease the safety of the original structure. The upper structure in the new spillway is made heavier than in the original dam.

For each horizontal section the water loading is exactly the same in the new and in the old structure. Thus, either the original structure is insufficient or the excess of the new is useless. In any case, the subjection of the original structure to greater loadings and unit stresses than those for which they were evidently designed is dangerous; 22 000 lb. per sq. in. = 15.5 kg. per sq. mm. in the steel, and 800 lb. per sq. in. = 56.5 kg. per sq. cm. in the concrete, are too much, at least, according to European provisions. The paper does not give the necessary data with which it would be possible to compute the unit stresses in the structure and on the soil, but the writer is sure that the load on the soil at the down-stream foot of the dam is excessive. Since, under favorable circumstances, a part of the original structure remained intact, it was advisable to maintain the original maximum water level.

The allowed over-topping of 2½ ft. of water from a height of 40 ft. between Buttresses 19 and *A* is, it would seem, also dangerous. The shock of the falling water will not only break the provided horizontal and inclined reinforced mat, but, when that occurs, it will wash the soil from under the dam footings.

On page 917, Mr. Scheidenhelm, criticizing the rupture of the original dam, says: "The conditions obtaining at the Stony River dam site are such that a safe dam could be built there. * * * The dam

should, and could, have been designed and constructed so as to be absolutely safe." Mr. Rutenberg.

The writer believes the reconstructed dam is less safe than the original structure.

The flaws in the foundation soil which had not been taken care of by the constructors of the original dam, widened and became more pronounced by the water pressure formed by the old dam. After the rupture occurred, some of these flaws came to light and could not but be remedied; so that the foundation soil was relatively reinforced, thus giving it another short lease of life, longer than for the original dam.

The writer is firmly convinced, however, that were the reservoir now to be emptied and the foundation soil examined it would be found to be in a very bad state.

"Certain of the drainage openings yielded muddy water, instead of the normally clear water. * * * Such openings were grouted shut, rather than take any chances of harmful erosion under the footings." (Page 1000.)

This "harmful erosion" has surely found other ways, unobserved by the author.

The writer is firmly convinced that the reconstructed dam will also be broken, perhaps soon, if adequate repairs are not made in time. It is a question of preventing a new disaster. These are the writer's reasons for taking the liberty of criticizing the work done in the reconstruction of the Stony River Dam.

Fred F. Moore,* M. Am. Soc. C. E. (by letter).—After study of this excellent description of the interesting reconstruction of an uncommon type of dam, two impressions stand out prominently: Mr. Moore.

First, that there is a dearth of information regarding sub-surface conditions as a guide to the original designs. Even now, comprehensive knowledge of the character of the foundation materials, and the behavior of these materials under the conditions imposed, does not appear to be sufficient to justify the conclusions reached by the author as to the margins of security of the reconstructed dam. Tests of frictional resistances of soils, as carried out in connection with this work, are of doubtful value as an aid to the judgment, because of the small areas in contact, and the values of frictional resistance adopted for the design seem to be considerably larger than even the indications of these experiments would justify.

Second, with reference to the function of the walls at "heel" and "toe" in resisting movement of the structure as a whole, the analyses which consider sliding and overturning independently would appear to befog rather than to aid the judgment. Materials such as these,

* New York City.

Mr. Moore. mixed with clay and saturated with water, perhaps under considerable pressure, and subjected to large loads, would flow, or, at least, transmit pressures in all directions like a liquid. In order that the cut-off wall at the "heel" may aid in resisting sliding as described, there must be a sensible movement of the structure as a whole, with consequent large uplift on the base of the dam. Hence, the sliding and overturning, or, perhaps better, the floating, of the structure must be considered with reference to the effect or reaction of these tendencies one on the other. That is, the materials under the dam, with reference to the function of the cut-off wall in resisting sliding, must be considered as a flowing medium rather than one in which a sliding may occur on certain planes like one solid on another.

The wall at the "heel" does not appear to have sufficient strength for the purpose contemplated. If this wall is fractured, water under reservoir pressure in considerable quantity would have access to the materials under the dam. The drainage provisions appear to be a doubtful reliance for minimizing uplift pressure. The danger of portions of the dam being floated off the foundations from leakage of water under pressure through the cut-off at the "heel", and through, or under, the cut-off at the "toe", at a time of large overflow, appears not to have had sufficient consideration. Although the idea of using the materials between the cut-offs and immediately down stream from the "toe" to resist sliding is a good one, it is doubtful if the reconstructed dam has a sufficient margin of security to warrant a conclusion that it is not likely to fail again.

The run-off assumed for the design of the spillway appears to be reasonable for the purpose, but the margin of 6 in. against overtopping is too small. A flow of 2 ft. of water over the intermediate spillway would not fall "well-nigh" vertically. Such a flow would strike the slope of the fill (Plate XII), which is inadequately protected for such a condition. Even water falling into the concrete channel from the top of the dam would start serious trouble through erosion of the bank by wash. Regardless of damage by falling water, the concrete channel is too small. Any considerable flow over the intermediate spillway would be almost certain to start cutting of the materials at the "toe".

The use of brittle steel pins for the flash-board support is unusual, and would seem to be an expedient of doubtful value.

Mr. Downs. W. S. Downs,* Esq. (by letter).—Ignorance of the coefficient of sliding friction between rock sub-strata, and the resulting neglect to provide the proper safeguards, have been the cause of numerous dam failures. Such failures never occur where the rock is of igneous origin, such as granite, trap, or the like, but authenticated records

* Morgantown, W. Va.

show that this danger lies in the softer sedimentary formations where the rock is horizontally stratified or nearly so. Mr. Downs.

Probably in no extensive geological formation is sliding or slipping of materials more in evidence than in the formations of the Carboniferous period, especially that portion of these formations which include the coal measures of the Appalachian fields and in which the Stony River Dam is situated.

These formations are highly stratified, the strata usually consisting of rather soft shales, ranging from a very soft fire-clay shale to a sandy shale, with occasional thin strata of coal and limestone and lentils of sandstone. One of the chief difficulties in studying the formations, however, is the non-uniformity and non-persistency in the character of the strata. The same stratum sometimes varies from a hard sandy shale to a soft fire-clay shale within a few hundred feet. The shales decompose and disintegrate very rapidly when exposed to atmospheric conditions, and, when decomposed, the resulting clays and soils are difficult to drain and exceedingly unstable.

The available data on the coefficient of static friction, especially as to layers of shales and clays, are extremely meager. In fact, very little has been learned on this subject since Morin made his experiments in 1834. The intensity of the pressure and the length of the time of contact between the two surfaces are factors the influence of which on the universal laws of friction are but vaguely understood. Therefore, until more extensive experiments have been made, and these general laws are more thoroughly understood, it behooves the engineer not only to make as thorough tests as practicable on the materials in question, but to use an ample factor of safety in designing.

Unfortunately, tests on a large scale are practically impossible, and engineers must often base their conclusions on tests of small areas of contact and with relatively light pressures. In applying these tests to the whole foundation area, they must depend largely on their knowledge of the geological conditions, coupled with an abundance of good judgment.

Mr. Scheidenhelm's tests on the frictional resistance of the foundation soils, as shown in Table 2, are interesting, and are valuable as an indication of the character of the materials in question. However, too much dependence should not be placed on the results of these tests. The same materials might show very different results when subjected to the more intense pressure of the dam for an indefinite length of time; and, moreover, the condition of saturation, as it must exist beneath the dam, might change the value of the coefficient of friction very materially. The writer has had occasion to study numerous slips and landslides in materials of this character, and has been surprised to note the vast difference in frictional resistance

Mr. Downs. between dry and wet clays. He has in mind one instance where dry clay soil indicated a coefficient of frictional resistance of more than 0.7, but after it had become thoroughly saturated for a considerable time, the coefficient of frictional resistance was less than 0.2.

Referring to the results of Mr. Scheidenhelm's tests, in Table 2, it will be observed that there is considerable difference in the frictional resistance between wet and moist samples of the same materials, and therefore the question arises: Might not the frictional resistance be still lower than is indicated by the tests when the clays are subjected to the conditions that will exist in the foundation materials of the dam?

At first thought, the coefficients of frictional resistance for shales (0.40 and 0.50), assumed by Mr. Scheidenhelm in the design for the reconstruction of the dam, may seem to be rather low. However, it should be remembered that the shales in question have "pronounced horizontal laminations" and are somewhat disintegrated, and that clay seams exist in places.

The lamination planes in the fire-clay shales of the coal measures often show a slick, smooth surface, sometimes of a more or less slippery nature. It is very evident that the cohesion between the substrata at these planes must be exceedingly slight, and, unless the planes of contact are very much warped or of limited extent, the resistance of the materials to sliding must be rather low.

Mr. Coburn. H. L. Coburn,* M. Am. Soc. C. E. (by letter).—The Engineering Profession is to be congratulated that there are engineers who have the time, the disposition, and, more especially, the ability, to prepare such a paper as this. One rarely sees a more complete exposition of an engineering problem from inception to finish than is here presented. The writer says from inception, for, to all practical purposes, this was an entirely new problem, as few or no data were available for the author's use.

As originally built the Stony River Dam was designed to meet certain stated conditions of foundation, which, subsequent events proved, did not apply, and the failure of the dam is attributable to this fact alone and not in any way to the type of structure. No dam designed to fulfill the conditions given could have withstood the treatment to which this one was subjected, and the writer thinks it an evidence of sound fundamental principle that the dam failed only in part and that part so gradually.

This is neither the time nor the place to discuss the conditions that led to the designing and building of this dam on data that were wholly inadequate and erroneous. The great lesson to be learned from the failure is the advisability, not to say necessity, of complete preliminary investigations on which to base design, and adequate and

* New York City.

competent engineering supervision of construction—money thus spent is "well spent." Mr. Coburn.

As to the reconstruction, though the writer doubts that all the safeguards against a possible second failure, which Mr. Scheidenhelm has taken, were necessary, yet he feels that, in the circumstances, they may be justified. If more accurate data were available as to "passive thrust" or resisting power of the local soils, and more real information as to coefficient of friction between these soils and the concrete cast thereon, engineers might criticize his details in some respects, but with the limited knowledge of actual values, and particularly in view of the fact that this was a case of rebuilding a structure which had partly failed, and about which very sensational stories had been circulated, the writer considers that if the dam was to be rebuilt at all, it was good judgment to assume the "worst possible conditions" and to make assurance doubly sure.

H. F. Dunham,* M. Am. Soc. C. E. (by letter).—In the discussion of a paper on the repairs made to a structure, it may not be in order to refer to the reasons for erecting that structure; but, if such reference is allowable, replies to two or three questions may be of interest. Mr. Dunham.

An article published in a technical journal† at about the date of the construction of the Stony River Dam contained a few words relating to the object to be secured by the improvement. At some point on the North Branch of the Potomac below its Stony River tributary there was a paper mill with an insufficient supply of wash-water in dry-weather periods. There was no mention of the use of an increased flow of the river for power purposes. There were no figures for the dry-weather discharge of the main stream at the mill, nor of its tributary. The additional quantity of wash-water needed was not given, and, furthermore, there was no longitudinal profile of either the North Branch or the Stony River, and no description of the river valleys or reference to any other method of providing the unestimated but desired supply.

It should be generally admitted that if, within a few years, the supply of wash-water has approached the desired quantity there would certainly be enough at all times, if the natural or original conditions pertaining to those streams could be restored. When the entire district was heavily wooded, the winter snows were longer in melting, the ground did not freeze to such depth as now, and more of the spring rainfall remained as ground-water for a longer period. Moreover, when floods were less frequent, fallen tree trunks and branches lodged in narrow channels, holding the water back. The beaver built his dams, with similar results, and when they were old he, too, repaired

* New York City.

† *Engineering News.*

Mr. Dunham.

them. All of this was favorable to a longer and more uniform flow of water in dry-weather periods than obtains at the present time. It may be within reason to inquire how far Nature's regulation of flow in such streams can now be imitated. The forest and frost are items that cannot be changed, but, given favorable topography and soil, the fallen timber dams and those made by the beaver with the higher ground-water table could be copied successfully and possibly improved.

The photographs in the paper, and the dam itself, 1 000 ft. long at the most favorable location, indicate a valley of some width. The sandstone mentioned, and the fact that the water found its way under the cut-off wall, show the existence of a somewhat porous soil. It is evident that the structure described was expensive. Had a part of that expense been used to construct a number of low but permanent dams at favorable places for low dams, each to be provided with a cheap low-water sluice-gate that would require a minimum of attention, a considerable change in the dry-weather flow of the stream would have been secured. The author refers to the existence of ground-water in and about the foundations of the dam. To raise the level of such ground-water over considerable areas gives available storage, and water thus stored is not subject to such rapid evaporation as surface water.

The dams, of course, would hold back surface water as well as ground-water. The extent to which the flow of a stream is regulated by higher ground-water level is often surprising. This is well illustrated by rivers flowing through wide flood-plains which, sponge-like, retain large quantities of flood water which are gradually returned to the river in its lower stages.

At certain seasons, especially in the spring, streams of moderate or sluggish current may carry too much silt to be suitable for a supply of wash-water. In the article previously referred to no mention was made of any filtration process, but, if that method is used, a reservoir below the mills suitable for the purpose of sedimentation and refiltration should have substantial value.

The writer has no wish to "go behind the returns" or to ask for any reference to the business affairs of business men that need not be disclosed. The first studies pertaining to any improvement are always interesting, and they are quite apt to be overlooked in the description of a finished work.

How the cost of the dam as it is now compares with the estimated cost of a gravity-section dam of the same general dimensions would be of interest, as would also the question of a curved versus a straight dam for structures of either type in that place.

Mr. Brodie.

ORRIN L. BRODIE,* M. AM. SOC. C. E. (by letter).—The author is to be thanked for the lucid and well-arranged presentation of the subject matter relating to the reconstruction of the Stony River Dam, and the thorough manner in which he has treated the various phases of that work.

One of the most striking features of his design, to the writer at least, was the unique method of anchoring the up-stream heel and cut-off. Relative to this matter, certain statements of the author in his brief outline of the previous failure of the structure were impressive. These were:

"Failure occurred where the up-stream cut-off wall extended only a short depth (5 to 7 ft.) into the over-burden."

"Failure was caused by undermining due to leakage."

"The type of dam was not in any way a cause of failure."

Thoughtful consideration of these three statements taken together forces on the writer the following query:

Would not a dam subject to considerable head of water, such as the one described, but containing sufficient mass that could contribute by its dead weight a resistance *per se* to sliding and overturning tendencies, be more satisfactory in every respect than one of the type here considered? Even though the hollow concrete type, such as the one described in the paper, may possess decided advantages in the matter of stability against overturning, there is always the possibility of the admission of water below it, which, besides increasing its tendency to slide, destroys any advantage inherent in its form in the matter of stability. Besides, as shown by the author, an up-stream cut-off thoroughly anchored to the heel seems to be a vital necessity. It is also significant that the reconstruction involved a quantity of masonry equal to that of the initial construction, which might better have been utilized in the first place.

The method of computing the storage effect of the reservoir area with respect to the spillway run-off was interesting to the writer, as was the detail of the spillway shape, as depicted on Plate X and noted by the author as being a more advantageous crest than that of the old spillway.

An investigation as to spillway run-off with respect to two basins, the waste weirs of which were at different crest elevations, was made by the writer, and involved an interconnecting tunnel between these two reservoirs (constituting the substitute supply works, connected with the new Kensico Reservoir for New York City).

The understanding of the method will be facilitated by the following nomenclature:

* New York City.

Mr. Brodie.

Volumes are in cubic feet; flows in cubic feet per second; and times, in seconds.

H_1', H_2', and H_1'', H_2'', are elevations of water surfaces in the respective reservoirs, Nos. 1 and 2;

t = time for Reservoir No. 1 to rise through a small distance (H' to H'');

V_1 = increment of capacity due to (H_1'' — H_1') for Reservoir No. 1;

V_2 = increment of capacity due to (H_2'' — H_2') for Reservoir No. 2;

I_1 = average inflow during time, t, from water-shed of Reservoir No. 1;

I_2 = average inflow during time, t, from water-shed of Reservoir No. 2;

i = average flow, during time, t, through tunnel and from Reservoir No. 1 to Reservoir No. 2;

Q_1' and Q_2' = flow over respective weirs at the beginning of time, t; and

Q_1'' and Q_2'' = flow over respective weirs at the end of time, t.

For Reservoir No. 1:

$$t = \frac{V_1}{I_1 - i - \frac{1}{2}(Q_1' + Q_1'')} \quad \ldots\ldots(1)$$

For Reservoir No. 2:

$$t = \frac{V_2}{I_2 + i - \frac{1}{2}(Q_2' + Q_2'')} \quad \ldots\ldots(2)$$

Eliminating i between Equations (1) and (2):

from Equation (1) $$t\left[I_1 - i - \frac{1}{2}(Q_1' + Q_1'')\right] = V_1 \quad \ldots\ldots(3)$$

from Equation (2) $$t\left[I_2 + i - \frac{1}{2}(Q_2' + Q_2'')\right] = V_2 \quad \ldots\ldots(4)$$

from Equations (3) and (4)

$$t = \frac{V_1 + V_2}{I_1 + I_2 - \frac{1}{2}(Q_1' + Q_1'' + Q_2' + Q_2'')} \quad \ldots\ldots(5)$$

Method, by successive approximations, of applying the foregoing expressions:

V_1 is a direct function of t and i, that is, $V_1 = f(t, i)$ and must be so considered.

Mr. Brodie.

Begin with Equation (5): First, assume values for V_1 and V_2 and corresponding values for the other terms in the right-hand member. Substitute the value of t thus found in the left-hand members of Equations (3) and (4), with the corresponding new value of i, until Equations (1) and (2) are satisfied. The second value of t can thus be tested by Equations (1) and (2). The values of V_1 and V_2 of Equations (3) and (4), respectively, must agree with the values of V_1 and V_2 in Equation (5).

The calculations, as briefly outlined in the foregoing, may be made in connection with the use of chosen run-off diagrams, together with capacity curves prepared both for the weirs, tunnel, and reservoirs. The results may be shown by appropriate curves of reservoir rise with time intervals from the beginning of the storm, or with respect to any stage desired.

In connection with the consideration of the spillway shape, the writer has determined the applicability of a parabola, the origin of which may be taken at the actual crest of the weir and the parameter at about 1.8 times the head of full reservoir level on the so-called "theoretical crest" (that is, the actual crest of a corresponding thin-edged weir). This parabola, with axis vertical and through the crest, will determine approximately the lower nappe or sheet of overfall for a weir.

The study leading to this conclusion was based on weir experiments by M. Bazin. This parameter may be increased for practical considerations to as much as 2¼ times the head.

The writer is not aware of the detailed method by which the shape of the new spillway crest of the Stony River Dam was fixed, but, by the foregoing method, he calculated the crest for the given head and was interested to find that, as far as could be ascertained by careful scaling of the spillway of Plate X, a practical identity resulted. The advantage of a larger value than 1.8 for the parameter factor used by the writer in this instance is that it permits of flattening the curve just down stream from the crest, thus improving the flow conditions. For a solid masonry weir, a larger parameter enables the parabolic section to be extended to a lower elevation than otherwise. In the case under consideration it was obvious that the comparatively steep apron slope required a minimum value for this particular parameter.

The author intimates that the preliminary soundings were made by test pits and auger borings, that failure occurred where the upstream cut-off wall extended only from 5 to 7 ft. into the over-burden, and that the technical advisers consulted on the original project were not familiar with the local conditions. Later, it appears, the results of core borings driven for reconstruction investigations disclosed

Mr. Brodie. lentils and alternations of pervious with impervious strata, and that sandstone boulders or false cliffs had been mistaken for bed-rock beneath the original cut-off construction.

These facts cannot emphasize too strongly the great importance of extensive sub-surface investigation preliminary to construction such as this, and the author's words in regard to foundation conditions as uncovered in the subsequent explorations, that they "required the most serious consideration and care", are conclusive.

Mr. Cain. WILLIAM CAIN,* M. AM. SOC. C. E. (by letter).—The writer is especially interested in the author's use of "anchoring walls" to increase the margin of security against the sliding of the dam. Such walls, projecting below the foundation, have been used repeatedly in dam and retaining wall design, but the projections were generally of small depth, so that little attempt has been made to state the principles affecting their design. The author's distinct contribution consists in the use of anchoring walls of such depth that the weight of the soil, from the foundation to the level of the lower end of the anchoring wall, is quite appreciable, and is utilized in increasing the resistance of the dam to sliding by lowering the possible plane of sliding from the foundation level to one passing through the bottom of the anchoring wall. He has entered into great detail regarding the principles affecting this design, and, in connection with it, has discussed the passive resistance of earth placed below the dam to sliding up a possible plane of rupture.

It is to be regretted that the author, in his experiments to determine the combined "adhesion" and friction of the various soils, did not apply increasing weights to the box containing the soil, and likewise measure the pull for each weight in turn, corresponding to "impending motion"; meaning by that term, that no actual sliding occurs, but that any increase in the pull, however small, would cause actual motion. If this had been done, the actual values of the coefficients of friction and cohesion could have been ascertained and used in the analysis. As the pulls were only recorded when the box containing the clay was in actual motion, the results give neither the friction coefficient alone nor the combined full cohesion and friction, such as would be actually exerted near the foundation of a stable dam, or perhaps on some lower plane or curved surface.

The results refer only to a combined friction and cohesion ("adhesion", as the author characterizes it, for this case), when the box was in motion. The numerical values of the "coefficients" in Table 2 are so much larger than the coefficients of friction of various clays, as given by Bell, as quoted in Table 7, that evidently a very appreciable amount of cohesion ("adhesion") was exerted during the motion. This

* Chapel Hill, N. C.

negatives the idea, suggested by the writer in a previous paper, that, after motion began, possibly all cohesion was destroyed, so that only friction remained to resist the pull. Evidently, from the results, although a large part of the full cohesion of the solid clay was not exerted during the motion, still an appreciable part of cohesion in addition to the friction was exerted during the actual sliding of clay on clay. Mr. Cain.

Coulomb's laws, concerning "impending" sliding of earth on earth, are symbolized in the equation:

$$Q = f P_n + c A \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

where Q = the total resistance to sliding in the plane of shear, in pounds;

f = the coefficient of friction;

c = the cohesion in the plane of shear, in pounds per square foot;

P_n = the normal pressure in the plane of shear, in pounds;

A = the area, in square feet, of the plane where shear is impending.

Also, if ϕ = the angle of friction, then f = tan. ϕ.

Table 7 gives experimental values of f, ϕ, and c, quoted from the writer's paper on "Cohesion in Earth"*, the first five values being from Bell, the last two from Jacquinot and Frontard.

TABLE 7.

Character of earth.	f = tan. ϕ, coefficient of friction.	ϕ	c, coefficient of cohesion, in pounds per square foot.
Very soft puddle clay, virgin state	0.000	0°	450
Soft puddle clay, virgin state	0.052	3°	670
Moderately firm clay, virgin state	0.087	5°	1 120
Stiff clay, virgin state	0.123	7°	1 570
Very stiff boulder clay, virgin state	0.287	16°	3 580
Sand-clay, very humid, kneaded	0.145	8°15′	885
Sand-clay, damp, virgin state	0.187	10°35′	443

The sand-clay referred to in Table 7 was taken from a well-rolled dam, made of the best materials, its composition being 60% clay, 32% silica as an impalpable dust, and 8% silicious sand.

Mr. Scheidenhelm's Table 4 furnishes the figures given in Table 8.

Since, in the author's experiments on the direct shearing of clay, the normal pressure, P_n, on the initial area was zero, it is seen, from Equation (1), that the last two columns give the coefficient of cohesion, c, in pounds per square inch and pounds per square foot, respectively.

* *Transactions*, Am. Soc. C. E., Vol. LXXX, 1916, pp. 1322, 1333.

Mr. Cain.

TABLE 8.

Character of soil.	Shearing value, in pounds per square inch of initial areas.	c, coefficient of cohesion, in pounds per square foot.
White clay	12.1	1 742
" "	26.6	3 830
" "	20.7	2 981
" "	26.0	3 744
" "	22.8	3 283
Black gumbo	9.6	1 382
"	16.1	2 318
Black loam	16.0	2 304
Sandy yellow clay	4.6	662
" " "	2.7	389
Average	15.7	2 260

It is seen, also, that the values of c compare very well with some of those in Table 7. Possibly, if the values of ϕ had been determined from appropriate experiments, they would also have compared favorably with the previous determinations.

It is important to observe that the time element affects all results very appreciably. Thus, Collin, in 1846, found that the resistance to shear in clay produced in from 12 to 15 min. was only one-third or one-fourth of that produced in from 12 to 15 sec. Bell states that all his tests were of considerable duration. This matter needs special attention by experimenters. As the author states that "the effect of the length of time of application of a given shearing load was not investigated," it is probable that the values of c, as given in Table 8, are all too large.

If certain values are assumed for ϕ, from Bell's experiments on clays as roughly applicable to the author's experiments relating to the resistance of clay on clay when in motion, a rude surmise may be made of the corresponding value of c for cohesion of motion.

Thus, for "white clay, fine-grained, wet", assume $\phi = 6°$; then, from Table 2, let $P_n = 200$ lb. and $Q = 0.34\ P_n = 68$ lb. With regard to A, the author says, "ordinarily, more than $\frac{1}{3}$ sq. ft. of the material under test was actually in contact." As definite information is lacking, it can only be assumed that $A = \frac{1}{3}$ sq. ft.; hence, by Equation (1),

$$68 = 200 \tan. \phi + \frac{1}{3} c,$$

or, for $\phi = 6°$, assumed, tan. $\phi = 0.105$. Therefore, $c = 141$ lb. per sq. ft. For the same "white clay, moist", from Table 2, if $P_n = 200$ lb., as an average, $Q = 0.42\ P_n = 84$ lb. Therefore, for $\phi = 6°$ (as before) and $A = \frac{1}{3}$ sq. ft., by Equation (1), it is found that $c = 190$ lb. per sq. ft.

Mr. Cain.

Again, for "yellow clay containing some grit, fairly wet", assume $\phi = 12°$ (tan. $\phi = 0.213$), $P_n = 200$ lb. Therefore, by Table 2, as an average, $Q = 0.68\ P_n = 136$. Taking $A = \frac{1}{2}$ sq. ft., by Equation (1), $c = 280$ lb. per sq. ft. Although the values of A and ϕ assumed are doubtless incorrect, the roughly approximate values of c, as computed, are instructive as showing a very appreciable cohesion, even when the upper layer of clay is in motion.

The author gives, on page 958, for certain conditions pertaining to the original dam before failure, the horizontal water pressure = 1 013 600 lb.; weight of dam and water over it = 2 030 400 lb.; both for a 15-ft. bay. If the base is 52 ft. wide, $A = 52 \times 15 = 780$ sq. ft.

Consequently, assuming the low value, $c = 660$ lb. per sq. ft. (say, for the yellow clay) and $\phi = 12°$ (tan. $\phi = 0.213$), on substituting in Equation (1), we find,

$$Q = 2\,030\,400 \times 0.213 + 780 \times 660 = 947\,300 \text{ lb.},$$

which is but little below the water pressure tending to cause sliding. In fact, the resistance to sliding is exactly equal to the water pressure for $\phi = 13°\ 50'$, which is a possible value.

This computation involves the assumption that there exists a cohesion between the concrete and clay at the foundation of 660 lb. per sq. ft., and the fact that the dam stood renders the assumption probable. No illustration can be given that more forcibly points to the need of "comprehensive experimentation" to determine the coefficients of cohesion and friction, not only for earth on earth, but likewise for concrete or other masonry on earth; for here is a dam that stood, which presumably should have failed if no cohesion was exerted at the base. This supposed cohesion that engineers (the writer included) have hitherto ignored, in computations affecting the sliding of dams and retaining walls, is probably a vital element concerning stability, and it doubtless has saved many walls from destruction by furnishing an additional resistance to sliding over that due to friction alone.

The argument that the resistance to sliding can be computed in terms of a friction coefficient alone, supposed to allow for the combined cohesion and true friction, is untenable, as the writer has conclusively shown.* The error can likewise be realized by a numerical example. Thus, on dividing Equation (1) by A, and putting $q = \frac{Q}{A}$ = unit total resistance to sliding, and $p_n = \frac{P_n}{A}$ = unit normal pressure, we have,

$$q = f\,p_n + c \qquad (2)$$

From Table 8, the lowest value of c, for the "sandy yellow clay" is

* "Cohesion in Earth", *Transactions*, Am. Soc. C. E., Vol. LXXX, pp. 1332–1334.

Mr. Cain.

389, say 400, lb. per sq. ft. On arbitrarily assuming $\phi = 12°$ or $f = \tan. \phi = 0.213$, Equation (2) reduces to,

$$q = 0.213\ p_n + 400.$$

Now, if it is assumed that a friction coefficient, $f = \frac{1}{3}$, with $c = 0$, can sufficiently represent the value of q, which denote by q', we have, from Equation (2),

$$q' = \frac{1}{3}\ p_n.$$

It will be found, for $p_n = 3\,333$ lb. per sq. ft., that $q = q'$; but, for $p_n < 3\,333$, $q' < q$; whereas for $p_n > 3\,333$, $q' > q$.

Thus, for $p_n = 1\,000$ lb. per sq. ft., $q' = 333$, $q = 613$ lb. per sq. ft.; but when $p_n = 6\,000$ lb. per sq. ft., then $q' = 2\,000$ and $q = 1\,678$ lb. per sq. ft.

The differences would be much more marked had large values of c been taken, corresponding to the "white clay" or "gumbo." As to the value $\phi = 12°$, arbitrarily assumed, it seems probable, from the experimental values of Bell, given in Table 7, that ϕ lies somewhere between 7° and 16°, and thus an average was taken as a probable value. Likewise, from the author's Table 2, the yellow clay shows a high, comparative value of the "coefficient of frictional resistance", including "adhesion", so that, possibly, ϕ should lie nearer to 16° than 7°, and 12° is possibly a little below the true value.

In connection with the foregoing values of p_n, it is of interest to know that, ignoring the cut-off wall and taking the area of the base of the dam as 52×15 sq. ft. for one 15-ft. bay, the average unit pressure on the foundation of the old dam is $\frac{2\,030\,400}{52 \times 15} = 2\,600$ lb. per sq. ft.; also, if the earth below it weighs 100 lb. per cu. ft., the unit pressure at 15 ft. below the foundation (at the bottom of the "anchoring wall", or at the level, *A B*, of Plate XII), amounts to 4 100 lb. per sq. ft. The values of q and q', as found from the foregoing equations, do not differ greatly for these values, 2 600 and 4 100 for p_n, but this is only due to the particular values of f and ϕ assumed.

It is not pretended that these are near the true values, as the computation was simply made to show what differences might occur by the two methods of estimating the resistance to sliding, and that the value of p_n, at the foundation or below it, is an important factor in the computation.

Although the data as to c and ϕ are "not sufficient in number and reliability" to lead to an exact determination of the passive resistance to sliding of this dam, by the theory of coherent earth, nevertheless, some interesting conclusions can be drawn, by finding, by this theory,

probable values of c and ϕ, necessary to the stability of the dam for each case of loading given by the author on pages 956-957. Mr. Cain.

It is very possible, as new experimental determinations come in, that surprises as to the true laws affecting friction and cohesion await us in the future, but, as far as the experiments go, Coulomb's two laws have been verified, and they will be made the basis of the following computations.

In the belief that the new method will prove of service, the analysis will be given in full for future reference, for immediate use by engineers who will determine experimentally the coefficients, c and f, for any soil in question.

The computation will refer to the revised section of the dam, Plate XII, but the cut-off wall will be ignored (which is on the side of safety), and the width of foundation (as estimated conservatively from the drawing) will be taken as 52 ft.

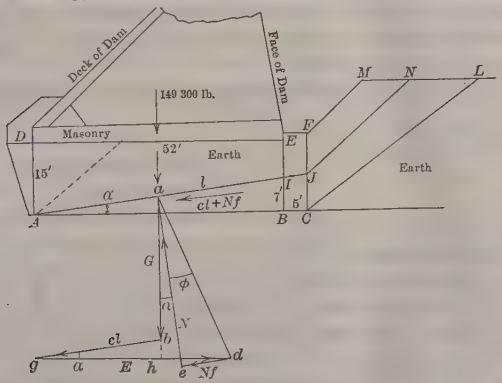


FIG. 28.

Thus, in Fig. 28, the width of foundation of the dam, $DE = 52$ ft.; the anchoring wall, or arm, extends from D to A, 15 ft., the wall being just to the left of AD. The toe-wall extends downward from E to I, 8 ft. To the right of this is earth, with the free surface, $EFMN$.

The analysis will proceed along the lines outlined by the author, but cohesion will be included and the resistance to sliding along IJ or BC will be ignored. The problem then consists of finding the horizontal component of the resistance to sliding along planes such as AI and JN or along AB and CL. Assuming the earth to weigh 100 lb. per cu. ft., the weight of earth, $AIED$, is 59 800 lb., which, added to the weight of the dam and of the water on the deck,

Mr. Cain. $2\,239\,400 \div 15 = 149\,293$ lb. per lin. ft. of dam crest, gives the weight on $A\,I = 209\,000$ lb. Denote this weight by G, the length, $A\,I$, by l, and its inclination to the horizontal by α. If $N = a\,e$ is the normal reaction of the plane, $A\,I$ (making the angle, α, with the vertical), then the resistance along the plane, acting to the left, to a horizontal force, E, tending to cause sliding along $A\,I$ to the right, is $c\,l + N\,f$. The friction component is $N\,f = \overline{de}$ and this, combined with $N = \overline{ea}$, gives a resultant, $\overline{da}$, making the angle, ϕ, with the normal $\overline{ae}$, since $f = \tan.\ \phi$.

Hence, if we lay off, to scale, $\overline{ab} = G$, then draw $\overline{bg} = c\,l$ parallel to $I\,A$; from g, draw a horizontal, $\overline{gd}$, to the intersection, d, with the line from a, making the angle, ϕ, above the normal $\overline{ae}$, then $\overline{gd} = E$, which is equal and opposed to the horizontal passive resistance to impending sliding up the plane, $A\,I$. This follows because the polygon of forces, $a\,b\,g\,d\,a$ or $a\,b\,g\,d\,e\,a$, is a closed polygon, where, $\overline{ab} = G$, $\overline{bg} = c\,l$, $\overline{gd} = E$, $\overline{de} = N\,f$, $\overline{ea} = N$, represent all the forces acting on the mass of earth, $A\,I\,E\,D$.

E can be found from the diagram or from an equation easily deduced.

Thus, $$E = g\,h + h\,d$$
$$= c\,l \cos.\ \alpha + (G + c\,l \sin.\ \alpha) \tan.\ (\alpha + \phi).$$
$$E = c\,.\,A\,B + (G + c\,.\,B\,I) \tan.\ (\alpha + \phi) \ldots\ldots\ldots\ldots (3)$$

It is evident from this equation,* regarding G as the load on $A\,I$ for any position of $A\,I$ between the one shown in the figure and $A\,B$, that E diminishes with α and is least for $\alpha = 0$, or along $A\,B$.

Now, to be conservative, take $\phi = 12°$ and $c = 300$ lb. per sq. ft., which is less than the smallest value (389 lb. per sq. ft.) given in Table 8 for the "yellow, sandy clay", and far less than the average, $c = 2\,260$ lb. per sq. ft. In fact, it is only slightly greater than the value (280) found for the yellow clay in motion. On substituting numerical values, $A\,B = 52$, $B\,I = 7$, $\alpha = 7°\ 30'$, $\phi = 12°$, $c = 300$, in Equation (3), we derive the resistance, exerted horizontally, to impending sliding up the plane, $A\,I$,

$$E = 90\,300 \text{ lb.}$$

To find the similar resistance to sliding along the horizontal plane, $A\,B$, first find the weight of earth, $A\,B\,E\,D = 78\,000$ lb., and add to 149 000 to get the G corresponding, $G = 227\,000$ lb. Then, making $\alpha = 0$ in Equation (3), or using Equation (1), we find the resistance to sliding along the plane, $A\,B$,

$$E = 227\,000 \text{ tan. } 12° + 300 \times 52 = 64\,000 \text{ lb.}$$

* On putting $(G + c\,.\,B\,I)$ in form $\left(227\,000 + A\,B \text{ tan. } \alpha \left(c \cdot \frac{A\,B}{2}\right)\right)$, the weight of dam, water over it, and earth, $A\,B\,E\,D$, being 227 000 lb.

If we assume the coefficients, c and f, the same along $D\,E$ (for concrete on earth) as above, or $c = 300$ lb. per sq. ft., $\phi = 12°$, we find the resistance to sliding along $D\,E$ to be, Mr. Cain.

$$149\,000 \times 0.213 + 300 \times 52 = 47\,000 \text{ lb.}$$

If it is recalled that it was found above, that the stability of the old dam was possible when $c = 660$ lb. per sq. ft., $\phi = 13° 50'$, then the values, c and ϕ, assumed along $D\,E$, appear to be conservative.

The horizontal component of the water pressure per foot of length of crest is $1\,218\,200 \div 15 = 81\,000$ lb. Of this, 47 000 lb. is carried by the earth directly down to the planes, $A\,I$ or $A\,B$, the remainder, 34 000 lb., being carried to the anchoring arm along $A\,D$, by which it is finally transmitted to the earth to the right of $A\,D$. The average pressure on the earth along $A\,D$ is $34\,000 \div 15 = 2\,267$ lb. per sq. ft., which is perhaps not so great as to cause flow, though the exact distribution of stress along $A\,D$ is unknown.

It is well, however, to enquire if this pressure of the arm, $A\,D$, may not cause sliding of the earth up some plane through A, shown by the dotted line. Hence the passive resistance of the earth to sliding up such a plane will next be investigated. Assuming that the reaction of the anchoring wall on $A\,D$ is horizontal, the unit horizontal passive resistance, a, of the earth at any point of $A\,D$ can be found from the equation,*

$$a = b \tan.^2 \left(45° + \frac{\phi}{2}\right) + 2\,c \tan. \left(45° + \frac{\phi}{2}\right),$$

where b = the weight, in pounds per square foot, acting on a horizontal plane at the point considered. As the distribution of stress on $D\,E$ is not given, assume it to be uniform, so that at D, $b = 149\,300 \div 52 = 2\,870$ lb. per sq. ft.; and at A, $b = 2\,870 + 15 \times 100 = 4\,370$ lb. per sq. ft. Hence, taking, as before, $\phi = 12°$, $c = 300$ lb. per sq. ft., the equation gives the unit passive resistance, acting horizontally to the left,

at D: 5 120 lb. per sq. ft.;
at A: 7 400 lb. per sq. ft.

As the flow of the earth is supposed to occur at 4 000 lb. per sq. ft., it is seen that sliding up the dotted plane will not occur when the maximum permissible stress on AD is taken at 4 000 lb. per sq. ft., as given by the author. If the permissible value to prevent flow had been assumed to be greater than 7 400 lb. per sq. ft., then the maximum unit compression at A must be taken at 7 400 lb. per sq. ft. or less; otherwise the earth at A would tend to slide up the dotted plane, which makes an angle, $45° + \frac{\phi}{2} = 51°$, with the vertical.

* Cain's "Earth Pressure", p. 192.

Mr. Cain. It now remains to compute the passive resistance, exerted on $F\ J$ or $F\ C$, of the earth to sliding up the proper planes of rupture, $J\ N$ or $C\ L$, corresponding. The graphical construction for effecting this will be given presently; but, to complete the present investigation, the result may be anticipated that the passive resistance on either $F\ J$ or $F\ C$ exceeds the safe compressive stress of the earth, so that the latter will limit the available resistance. The unit passive resistance at the free surface is not zero, but a very appreciable quantity, as will be shown eventually, and reasons will be given why an average pressure on $F\ J$ or $F\ C$ of 3 000 lb. per sq. ft. can be allowed without fear of the compressive strength of the earth being exceeded, or flow being imminent. Thus, the total compression, acting horizontally, that will be allowed on $F\ J$, 9 ft. in depth, is,

$$3\,000 \times 9 = 27\,000 \text{ lb.},$$

and on $F\ C$, 16 ft. in depth,

$$3\,000 \times 16 = 48\,000 \text{ lb.}$$

It has been shown previously, omitting any resistances along $I\ J$ and $B\ C$, that the horizontal component of the resistance to sliding along $A\ C$ is 64 000 lb., and along $A\ J$, 90 000 lb.; hence the total horizontal component of the resistance,

$$\text{along } A\ C \text{ and } C\ F = 64\,000 + 48\,000 = 112\,000 \text{ lb.}$$
$$\text{and along } A\ J \text{ and } J\ F = 90\,000 + 27\,000 = 117\,000 \text{ lb.}$$

Both of these totals exceed the horizontal component of the water pressure, 81 000 lb.; so that, if the assumed values of, $c = 300$ lb. per sq. ft. and $\phi = 12°$, are safe values, the dam is secure against sliding, for the conditions assumed, along the planes taken. The conclusion will not be altered if any plane intermediate between $A\ J$ and $A\ C$ is taken in place of either of these planes. From what precedes, it seems that $c = 300$ lb. per sq. ft. is a safe value, but there is uncertainty as to the value of ϕ. If we assume $\phi = 4°\ 23'$ (tan. $\phi = 0.0767 = f$), it will be found that, using $c = 300$, as before, the horizontal component of the water pressure is exactly equal to the resistance exerted along $A\ C$ and $C\ F$. For a less value of ϕ, sliding would occur. From Table 7, the value $\phi = 4°\ 23'$ is seen to be a barely possible one, though the probable value is greater; hence, with the incomplete data as to c and ϕ, it can only be asserted, for the loading of the author's Case I, that this investigation shows that the dam is most probably safe against sliding. The investigation for the loading under Case II, or "under the most severe conditions within limits of reason", page 957, is exactly similar to the foregoing, so that only results will be given.

The uplift water pressure was subtracted from the former load on the foundation, giving a net load, per foot of crest, on the foundation

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of 90 400 lb. Of course, this uplift pressure must not be taken in addition from the weights, $A\,I\,E\,D$ or $A\,B\,E\,D$, of earth, since, from the unit water pressure (say) just to the right of A, acting upward, is to be subtracted the weight of a column of water of height, $A\,D$, and horizontal section unity, which gives, precisely, the uplift at the level, $D\,E$; so that the weight of earth, $A\,I\,E\,D$ or $A\,B\,E\,D$, is not to be increased by the weight of water above $A\,I$ or $A\,B$, or diminished in any way, since the uplift pressure has already been subtracted from the load on $D\,E$. The total horizontal force acting on the dam, due to water pressure and ice, is 101 300 lb. per lin. ft.

By an investigation similar to the foregoing, it is found, for $c = 300$ lb. per sq. ft., $\phi = 12°$ and the compression on $C\,F$ limited to 3 000 lb. per sq. ft., that the total horizontal resistance along $A\,C$ and $C\,F$ amounts to 99 400 lb., and along $A\,J$ and $J\,F$ to 96 500 lb. However, for $c = 388$ lb. per sq. ft., $\phi = 12°$, the least resistance is equal to the water and ice pressure, 101 300 lb.; so that the dam is secure against sliding for either Case I or Case II, if $c = 390$ lb. per sq. ft. and $\phi = 12°$.

It is interesting to know the amount of security when $c = 660$ lb. per sq. ft., $\phi = 12°$. It is found, for these values, that the horizontal resistance exerted along $A\,B$ and $C\,F$ amounts to 118 000 lb., and along $A\,I$ and $J\,F$, to 116 000 lb. The latter value exceeds the water and ice pressure by 15 per cent. Since it was found above that the old dam was just stable for $c = 660$ lb. per sq. ft., $\phi = 13°\ 50'$, the values assumed, $c = 660$, $\phi = 12°$, seem to be not unreasonable. Again, for $c = 660$, $\phi = 12°$, the horizontal resistance to sliding along the foundation is, for Case II, $90\,400 \times 0.243 + 660 \times 52 = 53\,600$ lb.; hence the anchoring wall at $A\,D$ carries,

$$101\,300 - 53\,600 = 47\,700 \text{ lb.}$$

or, $47\,700 \div 15 = 3\,180$ lb. per sq. ft., average pressure.

As stated above, the resistance along BC (5 ft. in length), as well as that along IJ, was neglected, there being but little weight over parts of these surfaces; also, the cut-off wall was omitted, or it was supposed to be cracked at A. There is evidently some resistance from the omitted portions.

It must be distinctly understood that, although probable values for c and f for Cases I and II, have been found above, it is not asserted that these are the real values exerted. More extended experimental data as to these coefficients are required before definite results can be predicted.

It remains to find the passive resistance of the earth to the right of $C\,F$ or $J\,F$ to sliding up some plane, as $C\,L$ or $J\,N$. Although the assumption of a plane surface of rupture, as $A\,B$ or $A\,I$, for the earth under the dam, is doubtless near the truth, the same cannot be said for

Mr. Cain. the earth below the dam, of the irregular free surface, *F M L*. Theory shows that, except when the free surface is level, the surface of rupture is curved; so that when it is, of necessity, assumed to be plane, in the graphical construction that follows, the results must be treated as only an approximation to the truth, and a factor of safety must be used. Here, again, no experiments as to passive resistance have been made, so that we are absolutely in the dark as to the true surface of rupture and the resulting passive thrust.

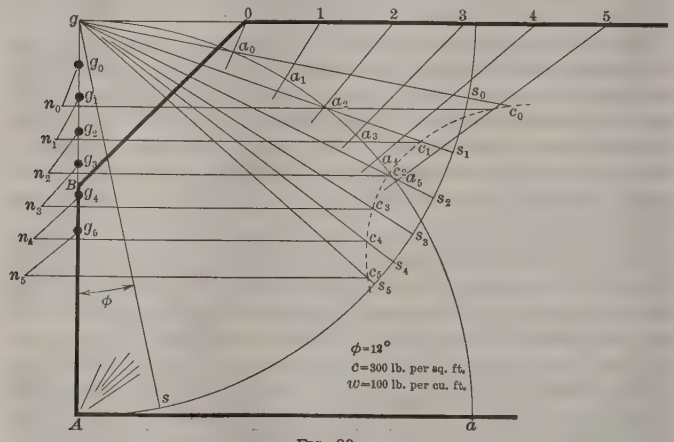

FIG. 29.

The graphical construction alluded to is given in Fig. 29, where $A\ B = 16$ ft., $B\ g = 11.5$ ft., and $g o = 11.5$ ft.; $A\ B$ (= $C\ F$ of Fig. 28) being vertical and $\overline{0\ 5}$ (= $M\ L$ of Fig. 28) being horizontal. It is desired to know the least force, acting horizontally to the right on $A\ B$, that will cause sliding up some plane of rupture $A\ 4$. Draw a number of trial planes of rupture, $A\ 0$, $A\ 1$, . . ., and draw $A\ a$ horizontal. Then, with A and some point, g, on the vertical, $A\ B$, produced, as centers, and with the same radius, $A\ g$, describe two arcs of circles, $a\ a_0$ and $A\ s\ s_0$. Assuming $\phi = 12°$, lay off $A\ g\ s = \phi = 12°$ and, with dividers, lay off the chord, $s\ s_0$ = the chord, $a\ a_0$, the chord, $s\ s_1$, = the chord, $a\ a_1$, etc., hence we have the equality of the angles, $s\ g\ s_0 = a\ A\ a_0$, $s\ g\ s_1$, $= a\ A\ a_1$, $s\ g\ s_2 = a\ A\ a_2$, etc. Then $g\ s_0$, $g\ s_1$, $g\ s_2$, . . . , make the angles, ϕ, above the normals to the planes, $A\ 0$, $A\ 1$, $A\ 2$, . . . , respectively. This is easily seen by referring to the force diagram of Fig. 28, in which it is observed that $a\ d$ makes the angle, $\phi + \alpha$, with the vertical. Similarly, in Fig. 29, if (say), the angle $a\ A\ 4 = \alpha$, then $A\ g\ s_4 = \phi + \alpha$, since we laid off

$s\ g\ s_4 = a\ A\ a_4 = \alpha$; hence, as in the case of $a\ d$ in Fig. 28, $g\ s_4$ makes the angle, ϕ, above the normal to the plane, A 4. It is convenient here to lay off the angle $\phi = A\ g\ s$ first, and then lay off the successive α's above $g\ s$. Mr. Cain.

Let it be assumed that the earth weighs 100 lb. per cu. ft. and that the coefficient of cohesion is $c = 300$ lb. per sq. ft.

The next step is to compute the weight of the successive trial prisms of rupture, $A\ B\ 0$, $A\ B\ 0\ 1$, $A\ B\ 0\ 2$, . . . , and lay off to scale on the vertical, $g\ A$, $g\ g_0$, $g\ g_1$, $g\ g_2$, . . . , to represent these successive weights. If, on the horizontal, $g\ 0\ 4$, we construct $0\ \overline{1} = \overline{1\ 2} = \overline{2\ 3}$, . . ., the triangles, $A\ 0\ 1$, $A\ 1\ 2$, . . . , are all equal in area, so that the total areas can be computed quickly by successive additions. The thickness of the prisms perpendicular to the plane of the paper is assumed as unity, so that areas and volumes are represented by the same numbers.

We next measure the lengths, $A\ 0$, $A\ 1$, $A\ 2$, . . . , to the scale of distance, and multiply the lengths, in feet, by $c = 300$, to find the successive cohesive forces acting downward (opposed to impending motion) along the planes, $A\ 0$, $A\ 1$, . . . , respectively. Lay off, to the scale of loads, $g_0\ n_0$, $g_1\ n_1$, . . . , parallel, respectively, to A 0, A 1, . . . , and equal to the cohesive forces acting along these planes. From n_0, n_1, . . ., draw horizontals to intersections, c_0, c_1, . . ., with $g s_0$, $g s_1$, . . .; then the smallest of the lengths, $n\ c$, to the scale of loads, will give the passive earth thrust on $A\ B$. In this figure, $n_4\ c_4$ is the shortest of the lines, $n\ c$, and it measures to the scale of loads, 69 300 lb., which is the passive thrust required, and A 4 is the corresponding plane of rupture. The method, just outlined, consists in treating in turn, $A\ B\ 0$, $A\ B\ 0\ 1$, . . . , as trial prisms of rupture and constructing the successive polygons of forces, $g\ g_0\ n_0\ c_0\ g$, $g\ g_1\ n_1\ c_1\ g$, . . ., similar to the polygon, $a\ b\ g\ d\ a$, of Fig. 28. The polygon, $g\ g_4\ n_4\ c_4\ g$, is found to have the least value of the thrust, $n_4\ c_4$, hence motion is impending for this thrust on $A\ B$, only along the plane A 4, since a greater thrust on $A\ B$, acting from left to right, is required to cause impending motion up the other trial planes of rupture. The forces in equilibrium acting on the true and only prism of rupture, $A\ B\ 0\ 4$, are, to scale, $g\ g_4$ = weight, in pounds, of $A\ B\ 0\ 4$, $g_4\ c_4$ = cohesion on A 4, acting down and parallel to A 4, $n_4\ c_4$ = thrust on $A\ B$, $c_4\ g$ = resultant of N and N tan. ϕ, where N = normal reaction of the plane, A 4, and N tan. ϕ is the friction acting down (opposed to impending motion) along the plane, A 4. The resultant of N and N tan. ϕ, thus makes the angle, ϕ, above the normal to A 4, as constructed in the beginning.

For the thrust, $n_4\ c_4$, on $A\ B$, acting from left to right, motion is impending not only along the plane, A 4, but upward along the plane,

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$A\ B$; hence, strictly, cohesion and friction are exerted downward on the plane, $A\ B$. Neglecting the cohesion, this would require that the lines of the type, $n\ c$, should be inclined below the normal to $A\ B$, or below the horizontal, at the angle, ϕ. Hence intersections, as c_4, will move down the corresponding $g\ s_4$, so that $n_4\ c_4$ and its horizontal component will be greater than before. The same is true for all the trial thrusts, of which the least is to be taken as the true one. The horizontal component of the true thrust, thus found, is greater than that found before, where the pressure on $A\ B$ was supposed to act horizontally.

From lack of any experimental data concerning passive thrust, it seems best, at present, to take the smaller thrust, corresponding to a horizontal pressure on $A\ B$, since the surface of rupture is curved and the assumption made that it is plane may possibly give results far from the truth. Hence, in computing the various values of the passive resistance to the push of the earth to the left of $C\ F$, Fig. 28, acting to the right, the push or thrust was taken as acting horizontally, as in Fig. 29.* For accuracy, other planes, lying near $A4$, should be tested in order to ascertain the minimum thrust represented by a line of the type, $n\ c$, or calculation can be resorted to. In fact, Equation (3), referring to Fig. 28, is directly applicable to each trial prism of rupture, provided G represents its weight.

Thus, in Fig. 30, let,

$G =$ the weight of the trial prism of rupture, $A\ D\ M\ I$;
$x = A\ B =$ the horizontal projection of $A\ I$;
$y = B\ I =$ the vertical projection of $A\ I$;
$\alpha =$ the angle, $B\ A\ I$;

whence tan. $\alpha = \dfrac{y}{x}$.

Then Equation (3) takes the form,

$$E = c\ x + (G + c\ y)\ \text{tan.}\ (\alpha + \phi) \ldots\ldots\ldots\ldots (4)$$

By the use of this equation, it is not necessary to draw Fig. 29 at all, as the equation can be applied to each trial prism of rupture in turn. Thus, for $\phi = 12°$, $c = 300$, for the plane, $A4$, of Fig. 29, $x = 31.5'$, $y = 27.5'$; therefore tan. $\alpha = \dfrac{y}{x} = 0.873$; therefore

* It may occur to one who has read pages 180–181 of the writer's "Earth Pressure", that, for the slope, FM, of Fig. 28, the theory pertaining to the uniform slope of indefinite extent, may be applicable; but, since FM is of limited extent, and the pushing force exerted on CF, from left to right, does not act parallel to FM, the conditions postulated for this theory do not obtain, and the theory is inapplicable. The surface, FM, makes an angle, $i = 45°$, with the horizontal, and if it was of indefinite extent, below and above F, slipping would occur at a depth, $x_0 = 7.62$ ft., when $\phi = 12°$, $c = 300$ lb. per sq. ft., by the theory alluded to, and the passive thrust, acting parallel to the surface, would be much smaller than that found above. If FM was large, say, 100 ft., at the same point of the slope, the theory might be approximately applicable; but, even then, the only effect would be a readjustment of the free surface contour to a new surface consistent with equilibrium.

$\alpha = 41° 07'$; also $G =$ the weight of $A\ B\ 0\ 3 = 36\,700$ lb.; whence, by Equation (4), $E = 69\,400$ lb. Mr. Cain.

Similarly, for the plane, $A3$, $x = 26.5$, $y = 27.5$, whence $\alpha = 46° 8'$; also $G =$ weight of $A\ B\ 0\ 3 = 29\,863$, and finally, $E = 69\,250$ lb. For a trial plane of rupture from A to a point midway between points (3) and (4), it is similarly found that $E = 69\,000$ lb.; hence this plane can be regarded as the true plane of rupture, and the thrust, 69 100 lb. (the least of the three), as very nearly the true thrust.

The passive thrusts for all the cases examined are as follows (referring to Fig. 28):

$c = 300$, $\phi = 12°$, passive thrust on $FJ = 34\,300$ lb.
" " " " $FC = 69\,100$ lb.
$c = 660$, $\phi = 12°$, " " " $FJ = 54\,800$ lb.
" " " " " $FC = 95\,000$ lb.

In the first case, N is to the left of M; for the other cases, both N and L were found to be to the right of M. These thrusts exceed the compressive strength of the material along FJ or FC, and hence they cannot be utilized in estimating the resistance of the dam against sliding, as previously noted.

It has been stated previously that the unit passive thrust at the free surface of the earth was not zero. To find approximately its amount at the point, D, let us find the thrust on a vertical plane extending 1 ft. below D, Fig. 30, by repeated use of Equation (4). The minimum value of E, which is the true thrust, is found to be 2 700 lb. per sq. ft. It corresponds to $\alpha = 61°$, nearly. By consideration of the total thrusts on the areas, FJ and FC of Fig. 28, it was estimated that the unit thrust at C was about double that at F. The unit thrust is not uniformly increasing, but varies about as shown by the little arrows in Fig. 30.

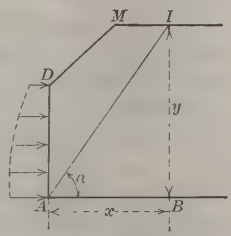

FIG. 30.

The unit compression on FC, Fig. 28, can be supposed to follow the same law, though, for simplicity, it can be regarded as uniformly increasing from a stress of 2 000 lb. per sq. ft. at F to double this, or the maximum allowable, 4 000 lb. per sq. ft., at C. This gives an allowable average stress on FC of 3 000 lb. per sq. ft., as used above.

It is seen from this investigation that the author is warranted in using a larger average safe stress on FC than the specified 2 000 lb. per sq. ft.

It may be observed that, if the force tending to cause sliding of the dam is entirely resisted along the plane, AB, then only the active

Mr. Cain.

pressure of the earth to the right of FC is exerted. If, then, the resistance along AB is supposed to diminish, first the active thrust of the mass, $CFML$, will be overcome and then more or less of the full passive thrust of this mass will be exerted, the amount always being less than that which would exceed the safe compressive stress of the earth, or would cause flow of the material along CF. Consequently, it does not appear that the passive resistance of any pile of earth placed below a dam to help resist sliding will ever come into play unless there is a slight movement of the dam down stream, since earth is not a rigid body.

A year or so ago, the writer's attention was called to the case of a high dam, where a large pile of earth was placed below and against the dam, to prevent possible sliding. He stated then that the coefficients of friction and cohesion were both needed to investigate fully the extra resistance to sliding supplied by the earth. He is glad of this opportunity to offer the full solution for such cases. Whatever the free surface contour, the graphical method of Fig. 29 offers a quick, approximate solution, though it is well, for the reasons stated, to apply a factor of safety to the result, or otherwise to use conservative values of c and f.

No better illustration could be given of the need of "comprehensive experimentation to determine the coefficients of friction and cohesion," than is afforded by this constantly recurring problem of estimating the resistance to sliding of dams and retaining walls. The writer is firmly convinced that a correct solution can only be attained by the use of the theory of coherent earth.

In conclusion, the writer wishes to state his appreciation of the very thorough and painstaking manner—down to the minutest details—in which the author has done the work of reconstructing the Stony River Dam.

Mr. Gregory.

CHARLES E. GREGORY,* ASSOC. M. AM. SOC. C. E.—A paper of this kind is unusually interesting, because it deals with the failure of a structure. The speaker believes that one can always learn very much more from the failure of a structure than from any discussion of what one believes ought to happen in accordance with theories.

The author has given a most lucid and well-written description of how he corrected the mistakes of the first dam, and has most ingeniously met the difficulties of the problem while utilizing the portions of the old dam which remained after the failure.

The reconstruction of this dam is an interesting subject, because it deals with a number of uncertainties pertaining to both the site and the type of dam, and uncertainties are always interesting. About the only relatively definite elements in the whole problem are the strength

* Mt. Kisco, N. Y.

of concrete and steel. Nearly everything else is open to more or less uncertainty. Notwithstanding this, the speaker believes all will admit that any large dam should be built so that its safety cannot be questioned, and one cannot say that any structure is safe until the uncertainties have been eliminated. Mr. Gregory.

In this case the type of dam and the site having been determined, there remain a great number of very uncertain features to be considered. The author has pointed out most of them, and has told how they were removed.

In any hollow dam, the uplift is the great doubtful feature and the great enemy of a dam of this type. There is no way of knowing to a certainty just what the uplift pressure is and how it is distributed. In designing a dam of this type, certain assumptions are made as to this pressure. It is assumed that if uplift exists, it can be controlled and limited by the weep-holes. Therefore, numerous weep-holes are placed in the foundation, and are supposed to relieve the upward pressure, and limit it to a relatively small amount. The cut-off wall must be effective, and must cut off the water from the impounded reservoir.

It is common experience that it is exceedingly difficult to build a cut-off wall, of any great depth and length, which will be absolutely water-tight. A concrete wall of great length will shrink and crack, in spite of anything that can be done to prevent it. Construction joints form weak planes, if not actual joints. All these conditions are favorable to leakage, even when one is successful in securing impervious rock foundation. Some water is bound to get through, and then it is bound also to show itself under the bottom of the dam.

When weep-holes are provided and are effective, they will allow this water to escape and relieve the pressure, but if they are not effective, the upward pressure will obtain. If they are effective, however, the water flowing through the soil under the bottom of the dam is a new source of danger.

A safe structure should not be menaced by flotation or by undermining. The flowing water from the weep-holes is certain to carry with it more or less soil, especially if it consists of fine, light grains; and undermining of the bottom of the dam will gradually occur. Eventually, the flow will become greater, heavier grains of soil will be carried, and ultimately there will be sufficient undermining to cause settlement and cracking, increasing the upward pressure so as to cause the dam to overturn or slide.

Probably this dam started to fail by overturning due to undermining. As soon as the tendency to overturn developed tension at the upper toe sufficient to open a joint and admit the water, the upward pressure floated the whole section off its base. The failure was probably a combination of overturning and sliding.

Mr. Gregory.

The speaker has not made any check computations whatever of the original structure, but from the author's statement of the various forces acting on this dam it would seem that, unless high coefficients of friction are assumed for the base, it would not have been safe from sliding. There seems to be no good ground for assuming high coefficients for a soil which is saturated with upward flowing water to the weep-holes.

In the speaker's opinion, the initial mistake was made in selecting a dam of this type for such a site. Although the speaker has not visited the site, and knows nothing about it, except what is disclosed in the paper, there appears to be in the vicinity considerable clay, gravel, and sandy clay which would be excellent material for an earth dam. An earth dam could have been built on this site economically, and would have eliminated nearly all the very uncertain features which obtain with this hollow concrete dam; the speaker believes an earth dam is the proper type for this site.

Another interesting part of the paper is the discussion of the spillway capacity. The author certainly has increased the capacity of the original spillway to a point beyond all precedent. The speaker believes that any spillway should have a very safe capacity. It should be large enough to care for the very largest storm that can possibly be conceived of, but the speaker believes that, in this case, the author has surely gone beyond the necessary limit. A flood as great as that provided for by the new spillway certainly could be produced only by a rainfall far greater than the maximum curve of rainfall for the eastern United States, as indicated by A. N. Talbot, M. Am. Soc. C. E. This curve is supposed to cover the greatest rainfall in this part of the country.

On page 930, the author cites certain storms of great intensity for 24 hours, but does not give the rate for the critical time for the water-shed. Probably a rainfall of 2 or 3 hours would be much more nearly the critical period for this water-shed. Fig. 31 and Table 9 present data from the automatic rain gauge in Central Park, New York, recorded continuously since 1869, and are self-explanatory. The maximum curve for Greater New York gives rates materially outside the Talbot curve, and is shown on Fig. 32.

On the illustration of the new spillway (Plate X) is shown a joint between the buttress wall and the slab covering the bays. This joint is filled with three-ply tar-paper. Apparently, such paper was not used in the old dam, or it would not have shown the bond that it did along those joints.

It would seem to the speaker that the placing of tar-paper in the joints of this dam, or in any dam, would be very poor practice. In the speaker's opinion, tar-paper could not be classed as a permanent material which would stand up under wet and dry, freezing and thaw-

Mr. Gregory.

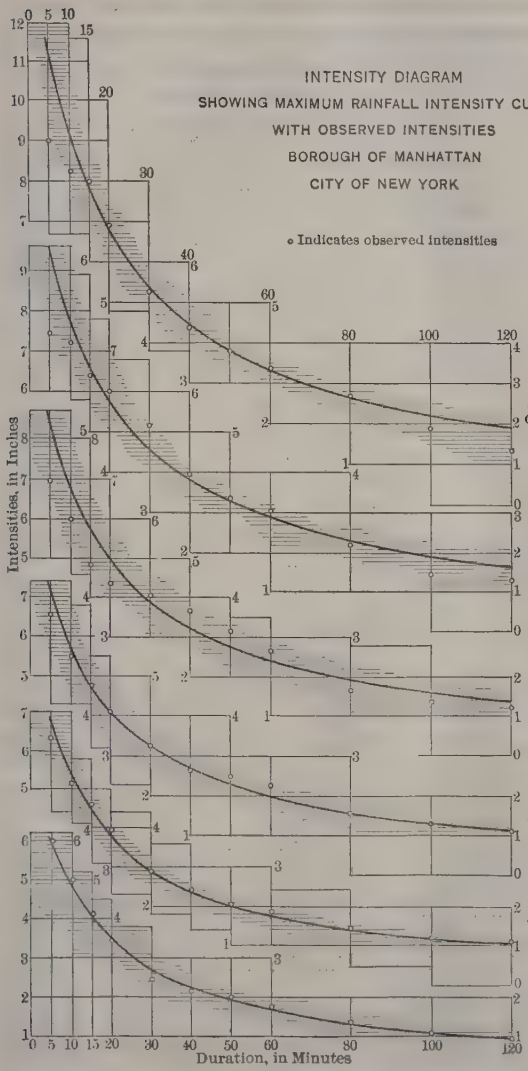

FIG. 31.

Mr. Gregory.

TABLE 9.—MAXIMUM RATES OF RAINFALL, IN INCHES PER HOUR, AS RECORDED BY RAIN GAUGE IN CENTRAL PARK, NEW YORK CITY.

Date.	Duration, in Minutes.										
	5	10	15	20	30	40	50	60	80	100	120
Aug. 5, '78	9.00	....	....	....	....	....	....	....	....	....	....
July 3, '92	7.44	....	....	....	....	....	....	....	....	....	....
July 31, '10	6.98	....	....	....	....	....	....	....	....	....	....
May 30, '12	6.60	....	....	....	....	....	....	....	....	....	....
Oct. 11, '71	6.48	....	....	....	....	....	....	....	....	....	....
Aug. 6, '95	6.36	....	....	....	....	....	....	....	....	....	....
July 12, '80	6.00	....	....	....	....	....	....	....	....	....	....
Aug. 20, '93	6.00	....	....	....	....	....	....	....	....	....	....
July 28, '02	6.00	....	....	....	....	....	....	....	....	....	....
Aug. 5, '78	....	8.20	....	....	....	....	....	....	....	....	....
Sep. 4, '13	....	7.20	....	....	....	....	....	....	....	....	....
July 31, '10	....	6.60	....	....	....	....	....	....	....	....	....
July 28, '13	....	5.70	....	....	....	....	....	....	....	....	....
May 30, '12	....	5.28	....	....	....	....	....	....	....	....	....
July 28, '02	....	5.16	....	....	....	....	....	....	....	....	....
Aug. 5, '02	....	5.16	....	....	....	....	....	....	....	....	....
Aug. 20, '93	....	5.10	....	....	....	....	....	....	....	....	....
July 26, '75	....	5.00	....	....	....	....	....	....	....	....	....
Aug. 5, '78	....	....	8.00	....	....	....	....	....	....	....	....
Sep. 4, '13	....	....	6.40	....	....	....	....	....	....	....	....
July 28, '13	....	....	4.88	....	....	....	....	....	....	....	....
Aug. 20, '93	....	....	4.80	....	....	....	....	....	....	....	....
May 22, '81	....	....	4.68	....	....	....	....	....	....	....	....
July 28, '02	....	....	4.60	....	....	....	....	....	....	....	....
July 26, '75	....	....	4.56	....	....	....	....	....	....	....	....
Aug. 5, '02	....	....	4.52	....	....	....	....	....	....	....	....
May 30, '12	....	....	4.12	....	....	....	....	....	....	....	....
Aug. 5, '78	....	....	....	6.93	....	....	....	....	....	....	....
Sep. 4, '13	....	....	....	6.00	....	....	....	....	....	....	....
Aug. 20, '93	....	....	....	4.85	....	....	....	....	....	....	....
July 28, '02	....	....	....	4.20	....	....	....	....	....	....	....
Aug. 5, '02	....	....	....	4.05	....	....	....	....	....	....	....
July 28, '13	....	....	....	3.98	....	....	....	....	....	....	....
July 26, '75	....	....	....	3.81	....	....	....	....	....	....	....
May 22, '08	....	....	....	3.57	....	....	....	....	....	....	....
Oct. 1, '13	....	....	....	3.45	....	....	....	....	....	....	....
Aug. 5, '78	....	....	....	....	5.22	....	....	....	....	....	....
Sep. 4, '13	....	....	....	....	5.12	....	....	....	....	....	....
July 28, '02	....	....	....	....	4.08	....	....	....	....	....	....
Aug. 20, '93	....	....	....	....	3.30	....	....	....	....	....	....
July 28, '13	....	....	....	....	3.16	....	....	....	....	....	....
Aug. 5, '02	....	....	....	....	2.90	....	....	....	....	....	....
Aug. 19, '04	....	....	....	....	2.76	....	....	....	....	....	....
July 26, '75	....	....	....	....	2.54	....	....	....	....	....	....
May 22, '81	....	....	....	....	2.42	....	....	....	....	....	....
Sep. 4, '13	....	....	....	....	....	4.42	....	....	....	....	....
Aug. 5, '78	....	....	....	....	....	3.93	....	....	....	....	....
July 28, '02	....	....	....	....	....	3.69	....	....	....	....	....

TABLE 9.—(*Continued.*)

Mr. Gregory.

Date.	Duration, in Minutes.										
	5	10	15	20	30	40	50	60	80	100	120
July 28, '13	....	....	....	....	....	2.70	....	....	....	....	....
Oct. 1, '13	....	....	....	....	....	2.59	....	....	....	....	....
Aug. 20, '93	....	....	....	....	....	2.49	....	....	....	....	....
Aug. 19, '04	....	....	....	....	....	2.40	....	....	....	....	....
Aug. 5, '02	....	....	....	....	....	2.35	....	....	....	....	....
Sep. 23, '82	....	....	....	....	....	2.18	....	....	....	....	....
Sep. 4, '13	....	....	....	....	....	....	3.84	....	....	....	....
July 28, '02	....	....	....	....	....	....	3.32	....	....	....	....
Aug. 5, '78	....	....	....	....	....	....	3.17	....	....	....	....
July 28, '13	....	....	....	....	....	....	2.88	....	....	....	....
Oct. 1, '13	....	....	....	....	....	....	2.14	....	....	....	....
July 6, '96	....	....	....	....	....	....	2.04	....	....	....	....
Aug. 23, '97	....	....	....	....	....	....	2.04	....	....	....	....
Aug. 2, '93	....	....	....	....	....	....	2.00	....	....	....	....
Aug. 19, '04	....	....	....	....	....	....	1.99	....	....	....	....
Sep. 4, '13	....	....	....	....	....	....	....	3.40	....	....	....
July 28, '02	....	....	....	....	....	....	....	3.02	....	....	....
July 28, '13	....	....	....	....	....	....	....	2.68	....	....	....
Aug. 5, '78	....	....	....	....	....	....	....	2.66	....	....	....
Oct. 1, '13	....	....	....	....	....	....	....	1.90	....	....	....
July 6, '96	....	....	....	....	....	....	....	1.85	....	....	....
Sep. 23, '82	....	....	....	....	....	....	....	1.74	....	....	....
Aug. 23, '97	....	....	....	....	....	....	....	1.74	....	....	....
Aug. 19, '04	....	....	....	....	....	....	....	1.72	....	....	....
Sep. 4, '13	....	....	....	....	....	....	....	....	2.70	....	....
July 28, '13	....	....	....	....	....	....	....	....	2.20	....	....
Oct. 1, '13	....	....	....	....	....	....	....	....	1.65	....	....
Sep. 23, '82	....	....	....	....	....	....	....	....	1.62	....	....
Aug. 19, '04	....	....	....	....	....	....	....	....	1.47	....	....
July 6, '96	....	....	....	....	....	....	....	....	1.46	....	....
May 21, '83	....	....	....	....	....	....	....	....	1.44	....	....
July 5, '01	....	....	....	....	....	....	....	....	1.36	....	....
June 29, '03	....	....	....	....	....	....	....	....	1.36	....	....
Oct. 1, '13	....	....	....	....	....	....	....	....	....	1.88	....
July 5, '02	....	....	....	....	....	....	....	....	....	1.44	....
Sep. 23, '82	....	....	....	....	....	....	....	....	....	1.36	....
Oct. 9, '03	....	....	....	....	....	....	....	....	....	1.29	....
Aug 19, '04	....	....	....	....	....	....	....	....	....	1.23	....
Oct. 4, '77	....	....	....	....	....	....	....	....	....	1.20	....
Sep. 4, '78	....	....	....	....	....	....	....	....	....	1.20	....
June 29, '03	....	....	....	....	....	....	....	....	....	1.20	....
Oct. 23, '12	....	....	....	....	....	....	....	....	....	1.13	....
July 5, '01	....	....	....	....	....	....	....	....	....	....	1.32
Sep. 23, '82	....	....	....	....	....	....	....	....	....	....	1.31
Oct. 9, '03	....	....	....	....	....	....	....	....	....	....	1.21
Sep. 4, '78	....	....	....	....	....	....	....	....	....	....	1.10
Oct. 4, '77	....	....	....	....	....	....	....	....	....	....	1.08
Aug. 19, '04	....	....	....	....	....	....	....	....	....	....	1.07
Aug. 4, '88	....	....	....	....	....	....	....	....	....	....	1.02
Aug. 23, '97	....	....	....	....	....	....	....	....	....	....	0.96
Aug. 23, '12	....	....	....	....	....	....	....	....	....	....	0.96

Mr. Gregory.

ing, as should have been provided in a permanent structure. These joints, though they should have been made free to move, should have had some more permanent methods of stopping leakage. Even a plain, smooth joint, without any filling, would have been far better than the tar-paper. Some form of metal tongue across the joint would have been still better.

When designing the Ashokan and Kensico Dams, of the Catskill Water Supply System, the speaker had occasion to divide these gravity masonry dams into sections for the purpose of taking up the contraction due to temperature changes. In the Ashokan Dam the joint was formed with concrete blocks, as shown on Fig. 33, and the leakage through the opening of the joint was prevented by constructing a 6-in. offset, with a face very accurately finished in a plane parallel to the longitudinal axis of the dam, so that when the joint opened the concrete surfaces would slide on each other without opening.

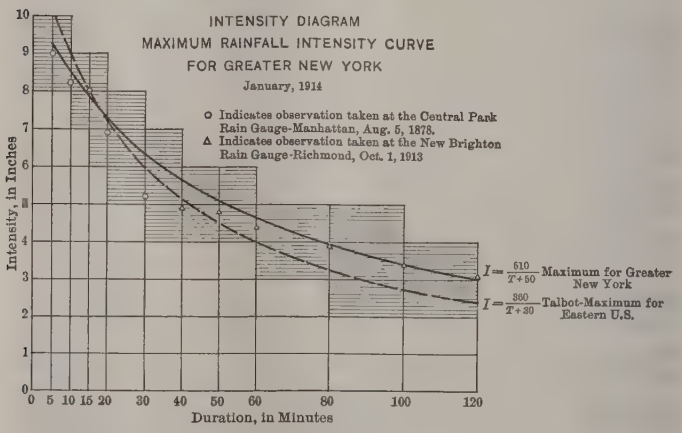
INTENSITY DIAGRAM
MAXIMUM RAINFALL INTENSITY CURVE
FOR GREATER NEW YORK
January, 1914
o Indicates observation taken at the Central Park Rain Gauge-Manhattan, Aug. 5, 1878.
Δ Indicates observation taken at the New Brighton Rain Gauge-Richmond, Oct. 1, 1913
Intensity, in Inches
Duration, in Minutes
0 5 10 15 20 30 40 50 60 80 100 120
0 1 2 3 4 5 6 7 8 9 10
$I = \frac{610}{T+50}$ Maximum for Greater New York
$I = \frac{360}{T+20}$ Talbot-Maximum for Eastern U.S.

FIG. 32.

Just down stream from this offset, as shown, is the drainage well to collect and carry to a lower gallery whatever leakage might come through the joint. Experience has shown the leakage through the fifteen joints of this dam, aggregating about 1 500 lin. ft. of exposed joint, to have been about 450 000 gal. per day as a maximum, and to have been reduced to about 25 000 gal. per day on November 2d, 1916.

In the Kensico Dam, similar joints were provided with a copper strip arranged diagonally across the sliding joint, so that, when the

Mr. Gregory.

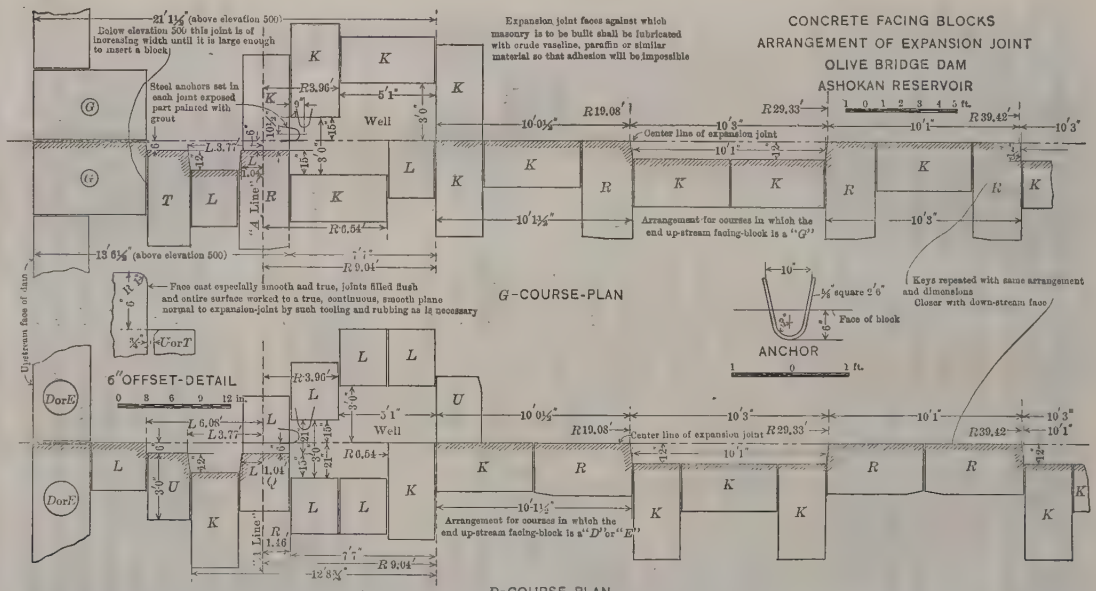
CONCRETE FACING BLOCKS
ARRANGEMENT OF EXPANSION JOINT
OLIVE BRIDGE DAM
ASHOKAN RESERVOIR
Expansion joint faces against which masonry is to be built shall be lubricated with crude vaseline, paraffin or similar material so that adhesion will be impossible
21' 1½" (above elevation 500)
Below elevation 500 this joint is of increasing width until it is large enough to insert a block
Steel anchors set in each joint exposed part painted with grout
Well
Center line of expansion joint
Arrangement for courses in which the end up-stream facing-block is a "G"
13' 6½" (above elevation 500)
Face cast especially smooth and true, joints filled flush and entire surface worked to a true, continuous, smooth plane normal to expansion-joint by such tooling and rubbing as is necessary
G-COURSE-PLAN
Keys repeated with same arrangement and dimensions
Closer with down-stream face
Face of block
ANCHOR
Up-stream face of dam
6" OFFSET-DETAIL
Arrangement for courses in which the end up-stream facing-block is a "D" or "E"
D-COURSE-PLAN
This arrangement is typical of an expansion joint, the south side of which is carried up ahead of the north. Where the north side is carried ahead, the joint, well, and blocks from the start, would be arranged opposite hand
Joints are ¾" except where varied to adjust length of groups of blocks

FIG. 33.

Mr. Gregory.

joint opens, the copper strip (as shown on Fig. 34) will crumple rather than tend to pull out. These structures have been very effective in all joints but two where there was considerable leakage at first, due to a poor batch of concrete near the bottom of the well in one case, and a contraction crack below the bottom of the well and copper strips in two cases which admitted more water than the entire twenty-two joints in the dam. After these cracks were grouted the leakage was reduced to from 13 000 to 18 000 gal. per day for the 1 900 lin. ft. of exposed joint.

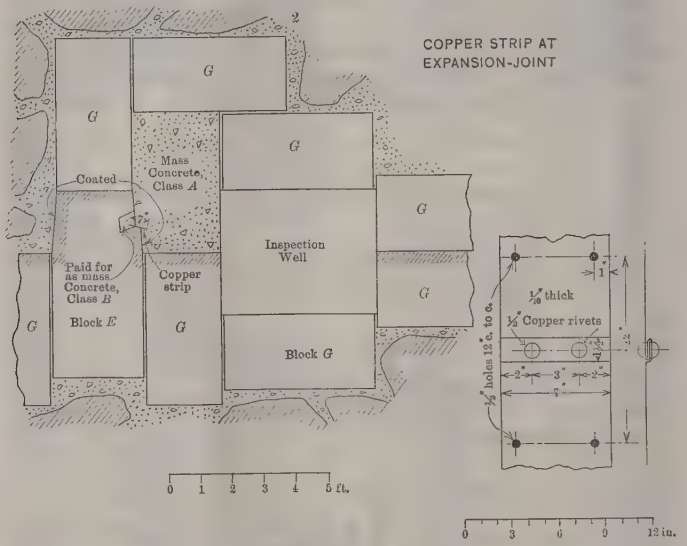

FIG. 34.

Mr. Grant.

KENNETH C. GRANT,* M. AM. SOC. C. E. (by letter).—This paper is very interesting to the writer, who made a careful examination of the dam on January 16th, 1914, and agrees with the author's conclusions as to its failure. The writer is impressed by the thoroughness of the studies and designs for the repair of the dam, and cannot but feel that it is unfortunate that the same care was not taken in the original design and construction, when equal safety could have been obtained at much less cost.

All readers of this paper must have been impressed by the large spillway capacity provided in the reconstructed dam. The writer was particularly pleased to see the retarding effect of the storage above

* Washington, D. C.

Mr. Grant.

spillway level worked out by actually routing the assumed maximum flood through the reservoir. The usual method of expressing spillway capacity, in terms of run-off per square mile of drainage area, does not tell the whole story. If the reservoir has a large surface area at spillway level, a rise of a foot in the water surface represents a very considerable storage capacity; and the maximum flood that can safely be taken care of by spillway and storage combined may be much greater than in the case of a reservoir with the same spillway capacity, but a small area of water surface. In the assumed maximum case shown on Fig. 15, the maximum spillway discharge amounted to about 1 600 sec-ft. per sq. mile, and the assumed flood causing this outflow reached a maximum of nearly 2 600 sec-ft. per sq. mile. Thus, the spillway capacity of 1 840 sec-ft. per sq. mile given by the author really means that it would take a flood about two-thirds larger than this to overtop the dam.

This reducing action, which every full reservoir exerts on a flood wave, is the same in principle as the control effected by retarding basins such as those which are to be built for the protection of the Miami Valley in Western Ohio, except that, in the latter case, the reduction is much greater, inasmuch as the entire capacity of the basins is available for flood storage. The writer has noted that the capacity of some of the flood-control reservoirs in Europe, in which the conduits can be closed by gates, is correctly considered to be the total capacity up to the elevation at which the spillway discharge reaches the maximum outflow that can be safely delivered to the channel below.

It may be of interest to explain briefly the method found to be the simplest for routing floods through the Miami Valley retarding basins, by applying it to the flood wave assumed by the author in Fig. 12. In the method here to be described, the time interval is fixed, and the rate of outflow is obtained by trial. The rate of inflow is fixed by the adopted time interval, and the trial rate of storage and elevation of reservoir surface are fixed as soon as the rate of outflow is assumed. As compared with the method of fixing the elevation of reservoir surface and finding the time interval by trial, this method has the advantage of enabling one to pick the breaks in the inflow curve, especially the peak, and the points where inflow equals outflow.

Fig. 35 shows a capacity curve of the Stony River Reservoir above spillway level, derived from Fig. 10; also a curve showing spillway discharges plotted against reservoir capacities, derived from Fig. 11. Fig. 36 shows the inflow and outflow curves, and Table 10 illustrates the method of recording the routing operation.

Conditions of starting similar to those given by the author, were assumed, namely, that, at 7 A. M., the outflow is equal to the inflow,

Mr. Grant.

or 100 sec-ft. From Fig. 35, this corresponds to a capacity of 208 400 000 cu. ft., or an elevation of 136.2.

Take a time interval of 1 hour. At this time the rate of inflow is 500 sec-ft. By trial, a rate of outflow at this time can quickly be

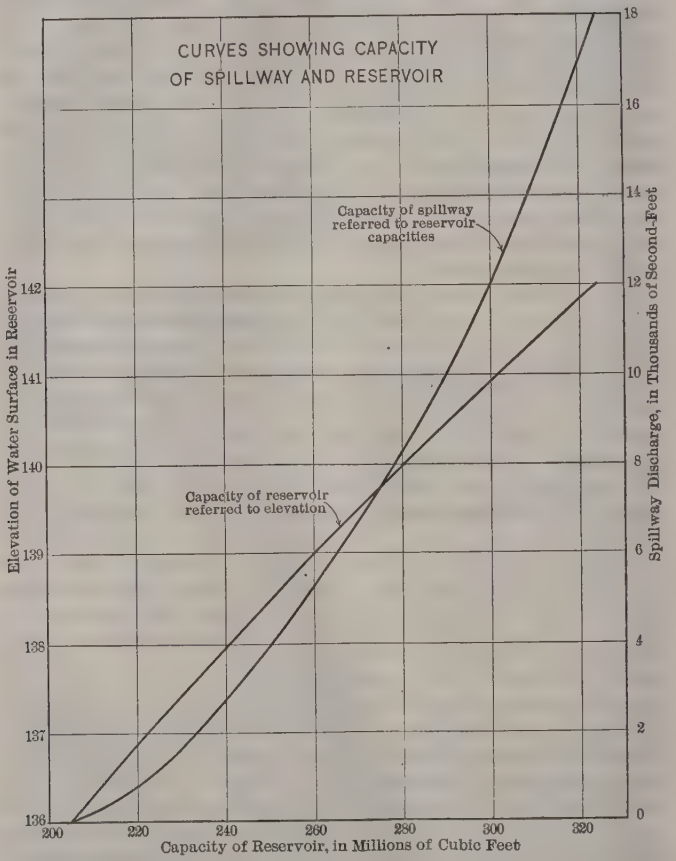

FIG. 35.

found such that the storage increment added to the storage already existing gives a total storage corresponding to the assumed outflow. Thus, by assuming a rate of outflow of 150 sec-ft. at the end of 1 hour, the rate of storage is 350 sec-ft., and the mean rate of storage is

175 sec-ft. Multiplying this by 3 600 gives the storage increment, in cubic feet, or 630 000 cu. ft. Adding this to 208 400 000, the storage at Elevation 136.2, gives 209 030 000 cu. ft., the total storage. Entering Mr. Grant.

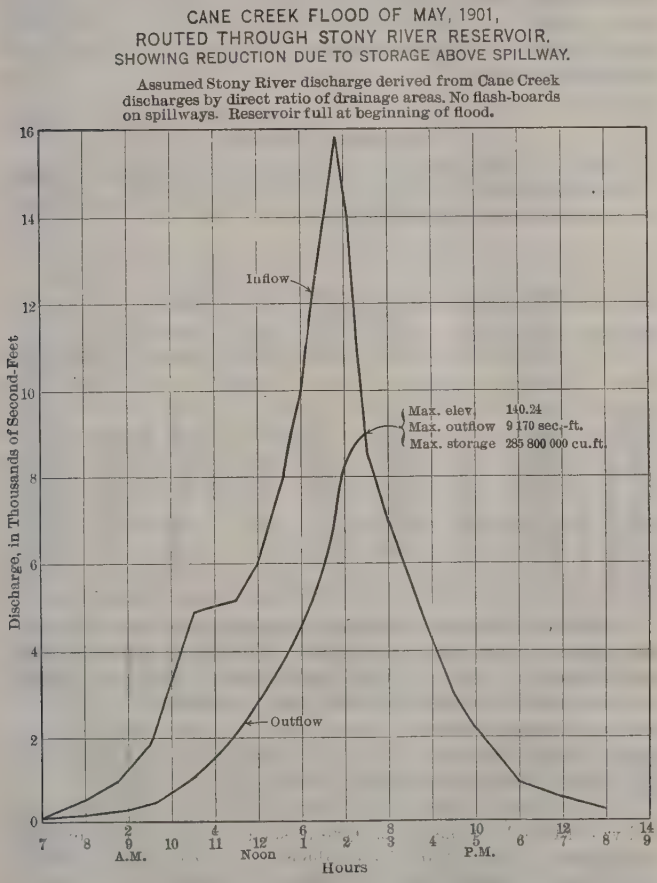

FIG. 36.

the spillway capacity curve on Fig. 35 with this reservoir capacity gives a discharge of 150 sec-ft., which checks the original assumption as to outflow. The elevation of reservoir surface, 136.24, is found by enter-

Mr. Grant.

ing the reservoir capacity curve on Fig. 35 with the total capacity, 209 030 000 cu. ft.

Continuing this operation, the maximum outflow is reached, at which time the outflow is equal to the inflow, and the rate of storage is zero. From this point until the water surface lowers to spillway level, the outflow is greater than the inflow, and the storage increments are negative.

TABLE 10.—CANE CREEK FLOOD OF MAY, 1901, ROUTED THROUGH STONY RIVER RESERVOIR.

Assumed Stony River discharges derived from Cane Creek discharges by direct ratio of drainage areas. No flash-boards on spillways. Reservoir full at beginning of flood.)

Elevation of reservoir surface.	Rate of inflow, in second-feet.	Rate of outflow, in second-feet.	Rate of storage, in second-feet.	Mean rate of storage, in second-feet.	TIME.		STORAGE.	
					Increment, in hours.	Total, in hours.	Increment, in millions of cubic feet.	Total, in millions of cubic feet.
136.20	100	100	0	0	0	0	0	208.40
136.24	500	150	350	175	1.0	1.0	0.63	209.03
136.38	1 300	270	1 030	690	1.0	2.0	2.48	211.51
136.55	2 000	420	1 580	1 305	0.6	2.6	2.82	214.33
137.04	4 850	1 080	3 770	2 675	0.9	3.5	8.68	223.01
137.70	5 100	2 250	2 850	3 310	1.0	4.5	11.91	234.92
137.98	6 000	2 900	3 100	2 975	0.5	5.0	5.35	240.27
138.40	8 000	3 820	4 180	3 640	0.6	5.6	7.86	248.13
138.98	11 800	5 250	6 550	5 365	0.6	6.2	11.60	259.73
139.77	15 800	7 550	8 250	7 400	0.6	6.8	16.00	275.73
140.05	14 000	8 480	5 520	6 885	0.25	7.05	6.20	281.93
140.24	9 170	9 170	0	2 760	0.39	7.44	3.88	285.81

This same method can be used, of course, where the reservoir is kept entirely empty for flood control, the spillway discharge curve being replaced by a curve in which the discharge of the outlet conduits is plotted against the corresponding reservoir capacities.

Mr. Jorgensen.

L. R. JORGENSEN,* M. AM. SOC. C. E. (by letter).—This is a very detailed analysis of the execution of a difficult piece of dam work—more difficult than the construction of the original structure.

To the writer it seems that an earth-fill dam with a concrete core-wall would have been the logical type in such a place, as there probably was enough clayey soil in the neighborhood to cover the bottom in an up-stream direction from the core-wall, although it is stated in the paper that there was not enough suitable material to build a plain earth-fill dam.

* San Francisco, Cal.

Mr. Jorgensen.

Of course, the author was not concerned with what could have been done in the first place when he was confronted with the task of repairing and strengthening the dam.

The author makes extensive studies for spillway provision, and arrives at a high run-off figure which seems to be justified. The breakable flash-board support arrangement is a good feature, considering the fact that the treated pins were found to break within narrow limits of load, that is, within between 4 and 4.6 ft. of head above the base of the flash-boards.

Valuable information as to the frictional resistance and shearing value of clayey soil and shale under various conditions is given in the paper. The anchoring wall at the heel and the work of tying it into the dam must have been among the most difficult details of the construction.

The stresses used, 700 and 500 lb. per sq. in., are somewhat higher than those to which the writer is accustomed, but, with good concrete, should leave sufficient margin for safety.

The drainage system has evidently been a problem of no small magnitude in this case, where the foundation is of such a character that muddy water is likely to continue flowing through the drains where these are placed very close to the reservoir pressure. The fact that the vertical drain pipes only fill themselves, but do not overflow, seems to indicate that there could not be very much uplift pressure, although the writer does not understand why these pipes just fill but do not overflow.

It is specified under Class *C* concrete that boulders should constitute not more than 40% of the total volume, and it would be of interest—at least to the writer—to know if the quantity actually placed in this concrete approximated this figure, as he has always felt that 20% of "plums" was about the maximum that could be "thrown in", unless there was considerable hand work. Concrete containing many "plums" is not likely to be as water-tight as when a medium percentage is used, say, 20%, which is not so small a percentage at that.

Mr. Wegmann.

EDWARD WEGMANN,* M. AM. SOC. C. E.—The speaker has been greatly interested in this description of the repair and reconstruction of the Stony River Dam. The conditions were very unfavorable, and the author is to be complimented on the very thorough manner in which the work was done, giving the dam large factors of safety against all possible kinds of failure which might occur.

In order to correct some erroneous statements which have been made—some of them in print—the speaker must state his connection with the inception of this dam. He was engaged by the West Virginia Pulp and Paper Company, in the fall of 1911, to visit the site of a

* New York City.

Mr. Wegmann.

proposed storage reservoir in the mountains of West Virginia, which site had been selected by an engineer employed by the Paper Company for this purpose. Owing to the steepness of the river beds in that region, it was very difficult to find a suitable site for a storage reservoir, and the one finally chosen, after considerable investigation, appeared to be about the only one available. The speaker was requested by the Paper Company to give his opinion about this site, and to recommend the type of dam which he thought should be adopted.

In accordance with this request, he made one visit to the site of the reservoir, late in the fall of 1911, in the company of Mr. R. P. Bloss, the engineer of the Paper Company. At that time, only a few test pits had been excavated. They showed that the material overlying the rock consisted of yellow clay mixed with very fine sand, and underlain by compact blue clay. The rock surface was found to be at a considerable depth. The surface of the ground was covered with numerous boulders, and in most of the test pits boulders were found at a certain depth below the surface. An alarming feature was the fact that veins of some fine black material which looked like coal appeared in some of the test pits. Analyses, made later, proved that this material was not coal.

Both Mr. Bloss and the speaker were much impressed by the possibility that leakage under the proposed dam might occur through the black veins in the clay, unless a proper cut-off was provided to rock bottom, either by a masonry wall or by sheet-piling. The speaker requested Mr. Bloss to dig additional test pits and to make soundings with an auger, in order to determine, as nearly as possible, the position of rock bottom. This work was done subsequently, and, early in 1912, Mr. Bloss submitted to the speaker a cross-section of the valley of Stony River, showing the probable line of rock bottom.

Based on this cross-section and the data obtained by the test pits and borings, the speaker reported to the Paper Company as follows:

1.—That the site selected for the reservoir would be satisfactory, if the valley were closed by a well-constructed dam.

2.—That a masonry dam would cost more than $500 000, owing to the great depth to rock, which is about 45 ft. in the center of the valley.

3.—That an earth dam was out of the question, as no earth or gravel, for mixing with the clay at the site of the dam, could be found within a reasonable distance from the proposed reservoir.

4.—That the only type of dam which could be built at the proposed site, within a reasonable sum—say, from $150 000 to $200 000—was a hollow dam of reinforced concrete.

Mr. Wegmann.

After receiving the speaker's report, the Paper Company decided to construct a hollow dam of reinforced concrete, and requested the speaker to prepare a contract and specifications for this work. Three different companies which had had experience in this kind of construction were invited to submit plans and bids for the construction of a hollow dam of reinforced concrete, based on the cross-section of the valley prepared by Mr. Bloss. On this plan Mr. Bloss, in consultation with the speaker, had marked the least depth to which the foundation of the cut-off wall would probably be excavated, and the contract provided that if the foundation should go deeper, the additional work involved should be paid for as an extra. At both ends of the dam, where it was thought that the cut-off wall would probably not go down to rock, sheet-piling was shown.

Four different plans* for constructing the dam of reinforced concrete were received, with lump-sum bids, ranging from $143 000 to about $200 000, for building the dam to the depth shown on the cross-section of the valley.

Only one among the bidders, the Ambursen Hydraulic Construction Company, of Boston, Mass., had had much experience in the construction of dams of the proposed type. This Company had built, at that time, more than seventy dams of this kind, ranging in height up to 150 ft. All these dams were standing, and although one of them—that at Pittsfield, Mass.,† the cut-off wall of which had not been carried deep enough in a foundation of gravel—had been undermined at the center of the valley, the dam had merely sagged, but had not been ruptured. As the hole made under this dam by undermining had been 20 ft. deep, 53 ft. wide, and about 50 ft. long, both up stream and down stream, the structure had certainly shown remarkable strength. It had been jacked up, a deeper cut-off wall had been provided, and no further trouble had been experienced.

In view of the wide experience of the Ambursen Company in constructing hollow dams of reinforced concrete, the speaker strongly advised the Paper Company: (1) to engage the Ambursen Company to design the proposed dam; and (2), in case this Company should not get the contract for building the dam, to employ one of its experienced engineers or superintendents to be constantly on the ground while the dam was being built, in order to insure that the plans of the Ambursen Company would be properly carried out.

These recommendations were adopted by the Paper Company, and the speaker's connection with the Stony River Dam then terminated. The speaker had never seen the plans finally adopted, until after the partial failure of the dam in January, 1914.

* *Engineering News*, September 5th, 1912.

† *Engineering News*, April 1st, 1909.

Mr. Wegmann.

It appears that the westerly half of the dam was constructed by the Webber Construction Company, the lowest bidder, and the easterly half was built by the Ambursen Hydraulic Construction Company. According to the plans shown to the speaker by the Paper Company, after the partial failure of the dam, and according to various published accounts, the cut-off wall was only 5 ft. deep below the floor of the dam from the west end to Buttress 15, although the depth of water at the latter point was more than 25 ft. This part of the core-wall was founded on what appeared to be hard-pan, a tough clay which was very hard to pick but became soft after being under water. For the remaining length, the cut-off wall was carried down to what was thought to be bed-rock. As the usual rule is to make the depth of the cut-off wall at least half the depth of the water, the inconsistency of the manner in which the dam was constructed was apparent, and it is difficult to understand how the engineers in charge of the work could have been satisfied with such a shallow cut-off wall, especially as sheet-piling had been omitted at the ends of the dam, although they knew that there were porous seams in the clay formation.

After the partial failure occurred, the Ambursen Company issued a bulletin about "The Facts as to the Blow-out under the Stony River Dam at Dobbin, West Virginia." In this pamphlet the Company stated that the plans submitted to them by the speaker "showed sheet-piling under each edge as a cut-off", but that its Mr. Ambursen, after visiting the site of the dam, "expressed disbelief as to the possibility of driving sheet-piling, and recommended a cut-off trench to be carried down into sound material." The pamphlet continued as follows: "The test pits were at that time* examined jointly by the owner's engineer, their consulting engineer, Mr. Wegmann, and Mr. Ambursen."

The speaker cannot understand how this statement got into the pamphlet, as it is absolutely incorrect. He never met Mr. Ambursen and Mr. Bloss at the site of the dam, and, at the time mentioned, had no connection with the work.

A few days after the dam had been ruptured, Mr. A. G. Hillberg, representing the *Engineering Record,* visited the site. According to the published account of his observations,† the core-wall had only been carried down 5 ft. below the flooring of the dam, at the point where the break occurred, although the depth of the water in the reservoir at this point was more than 25 ft. Mr. Hillberg found, at the point of rupture, a "pervious seam of coal and sand with some clay as a binder, about 8 in. below the level at which the cut-off wall had been stopped." The seam was from ¼ to 6 in. thick, and about 4 ft. wide. This discovery gave a clear proof of the cause of the

* April, 1912.

† *Engineering Record,* January 24th, 1914.

failure of the dam, and as Mr. Scheidenhelm, also, has stated in his report to the Public Service Commission of West Virginia that "failure was caused by the undermining of the over-burden or soil under the up-stream cut-off wall", the speaker thinks that this question might be considered as definitely settled. Mr. Wegmann.

Mr. Scheidenhelm has not only repaired the breach in the dam, but has made a number of important changes in its design and construction which the speaker will discuss.

Increasing the Spillway.—As originally built, the spillway was only 150 ft. long, and 3 ft. deep below the crest of the main dam. In case of a severe freshet, the whole dam could have acted safely as a spillway, if a suitable apron had been constructed on the down-stream side.

Assuming the probable maximum freshet at 1 386 sec-ft. per sq. mile—the figure given by the author for Cane Creek—the bulkhead section of the dam, as originally built, would have had to pass a sheet of water about 2.5 ft. deep. In all probability, the dam would have been able to pass this water, but, as no apron had been provided for the bulkhead section, the latter would have been gradually undermined.

In reconstructing the dam, Mr. Scheidenhelm increased the spillway capacity to about 1 840 sec-ft. per sq. mile. Although this may seem unusually large, it made the dam very safe.

Increasing the Storage Capacity of the Reservoir.—By placing flash-boards on the two spillways provided in the reconstructed dam, Mr. Scheidenhelm raised the water level 3.5 ft., and thus increased the reliable storage capacity of the reservoir by 25 per cent.

Resistance to Sliding.—The author made some experiments on the frictional resistance of clay moving on clay, shale on shale, and concrete on shale. In all these experiments, the coefficient of friction appears to have been determined for materials in motion. The force required to start the motion is known to be much greater than that needed to keep a body moving. As sufficient data on these points are not yet available, the author acted wisely in adopting conservative figures.

The original dam had withstood successfully for about 65 days the pressure due to a full reservoir, before failure occurred on January 15th, 1915, and there was no indication that the dam had not sufficient stability against sliding. As the weepers in the floor of the dam had, doubtless, been closed by ice during this period, and as the reservoir had not, at that time, been made water-tight by silting up, the dam, in all probability, had been subjected to upward pressure.

Resistance to Overturning.—One of the advantages of a hollow dam of reinforced concrete, having its deck on an angle of 45°, is its stability against overturning. As the water rose in the reservoir and finally over-topped the crest of the dam, the line of pressure was

Mr. Wegmann.

first drawn up stream and then down stream, and intersected the base of the dam near its center when the water reached the highest flood level. The speaker thinks the original dam would not have failed by overturning. The provisions made during the reconstruction to increase the stability against sliding, at the same time, gave the dam a still larger factor of safety against overturning.

Extension of the Cut-Off Wall.—Mr. Scheidenhelm has extended the shallow cut-off wall near the west end of the dam to rock. His explorations by test pits and drill-holes showed that, in some places, where the cut-off was thought to have been founded on rock, it was really built on boulders, and permitted leakage under its base. In such cases, Mr. Scheidenhelm carried the cut-off deeper, and, in other places, he made, by a small V-shaped trench filled with concrete, a water-tight seal between the up-stream side of the cut-off wall and the bed-rock. The width of this wall—only 2½ ft.—did not seem to be sufficient for the maximum pressure it had to sustain, and, therefore, it is not surprising that Mr. Scheidenhelm found that in some places water leaked through the cut-off wall.

Miscellaneous Construction.—Mr. Scheidenhelm has strengthened the original footings of the buttresses, remedying faulty conditions in places by pressure grouting. He has also provided a proper drainage system for taking care of the leakage through the foundation soil, and has housed in the higher bulkhead portions of the original dam by curtain-walls and roofs, in order to prevent serious freezing in the drainage system, etc.

In conclusion, the speaker compliments the author on the very thorough manner in which he has repaired and improved the dam, and on the detailed account of this work which he has given in his valuable paper.

Mr. Church.

Irving P. Church,* Assoc. Am. Soc. C. E. (by letter).—In connection with Fig. 12, of his valuable and exhaustive paper, the author refers to a tentative process by which such a curve as *a-b-g*, or "Discharge over Spillways", is determined; that is, a curve showing the rate of discharge, Q, over the spillway as a function of the time; so that with Q known as a function of H, the depth on the spillway, it becomes possible to compute H for any epoch; this being a case where the rate of influx into the reservoir, or "flood discharge", is given as a function of time, in such a curve as *a-c-d*, or graph of flood discharge, in Fig. 12.

As a matter of this kind is very rarely treated in books on hydraulics, it may be of interest to consider the strict mathematical nature of such a problem, that is, where efflux takes place through an orifice or over a spillway from a very wide vessel or reservoir, simultaneously

* Ithaca, N. Y.

with an influx into the reservoir; and where it is desired to determine the value of the head, H (head on orifice or over crest of spillway), as a function of the time, t. Mr. Church.

At any instant of time, suppose the surface of the water in the reservoir to be rising, and assume the following notation:

A = the area of that surface, at any instant;
F = area of orifice, if one is used;
μ = coefficient of discharge of orifice;

and, if a spillway is used, let

b = length of crest; and
c = coefficient of discharge.

Also, let

Q = rate of efflux, in cubic feet per second, at any instant;
and Q' = rate of influx, in cubic feet per second, at any instant.

During any short interval of time, $d\,t$ (time-increment), H increases by $d\,H$, and the gain of the volume of water in the reservoir must be equal to the excess of influx over efflux during this time; that is,

$$Q' \,.\, d\,t - Q \,.\, d\,t = A \,.\, d\,H \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

(If the surface were sinking, we would have $-A \,.\, d\,H$ instead of $+A \,.\, d\,H$.)

Case I.—Let Q' be constant and the reservoir have vertical sides, with efflux taking place through an orifice; then A is constant. Let H_0 be the initial value of H, and let the constant, Q', be written in the form, $Q' = \mu\, F \sqrt{2\,g} \,.\, K^{\frac{1}{2}}$, where K is an ideal constant head, easily computed. Equation (1) now becomes:

$$d\,t = \frac{A}{\mu\, F \sqrt{2\,g}} \,.\, \frac{d\,H}{K^{\frac{1}{2}} - H^{\frac{1}{2}}} \ldots\ldots\ldots\ldots\ldots (2)$$

This is readily integrable by a temporary change of variable, $K^{\frac{1}{2}} - H^{\frac{1}{2}}$ being denoted by Z; so that $d\,H = -2\,H^{\frac{1}{2}} \,.\, d\,Z$, $= 2\,(Z - K^{\frac{1}{2}}) \,.\, d\,Z$.

Now, $H = H_0$ for $t =$ zero and, finally, after integration, the form is obtained,

$$t = \frac{2\,A}{\mu\, F \sqrt{2\,g}} \left[\sqrt{H_0} - \sqrt{H} + \sqrt{K} \,.\, \log_e \left(\frac{\sqrt{K} - \sqrt{H_0}}{\sqrt{K} - \sqrt{H}} \right) \right].$$

giving the time, t, as a function of H. If t is given and H sought, resort must be had to solution by trial. (By "$\log_e$" is meant Naperian logarithm.)

Case II.—Let the reservoir have a rectangular spillway on which initially (that is, for $t =$ zero), the head is H_0 though at any later instant it is H, the rate of efflux being then $Q = c \,.\, b \sqrt{2\,g} \,.\, H^{\frac{3}{2}}$. Let the

Mr. Church. rate of influx, Q', be constant, and let an expression be written for it in the form, $Q' = c \,.\, b \sqrt{2g} \,.\, K^{\frac{3}{2}}$, where K is an ideal constant head. The sides of the reservoir are vertical; hence A is a constant.

Assuming that Q' is greater than the initial value of Q, we have from Equation (1):

$$d\,t = \frac{A}{c \,.\, b \sqrt{2g}} \,.\, \frac{d\,H}{K^{\frac{3}{2}} - H^{\frac{3}{2}}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (4)$$

The integration* of this equation is quite roundabout, but finally† leads to the result

$$t = \frac{A\,K}{3\,Q'}\left[\log._e \left(\frac{\left[\sqrt{K} - \sqrt{H_0}\right]^2 \left(H + \sqrt{H\,K} + K\right)}{\left[\sqrt{K} - \sqrt{H}\right]^2 \left(H_0 + \sqrt{H_0\,K} + K\right)}\right)\right.$$

$$\left. - \sqrt{12}\,\tan.^{-1}\left(\frac{\left(\sqrt{H} - \sqrt{H_0}\right)\sqrt{12\,K}}{3\,K + \left(2\sqrt{H_0} + \sqrt{K}\right)\left(2\sqrt{H} + \sqrt{K}\right)}\right)\right], \quad (5)$$

in which, as before, $\log._e$ signifies "Naperian logarithm of"; and $\tan.^{-1}$ denotes "anti-tangent of", or "arc whose tangent is"; for example, $\tan.^{-1}\ 0.488 = 0.454$ (since $\tan.\ 26° = 0.488$, and 26° expressed in arc ("radians") is 0.454).

Case III.—Rate of influx, Q', not constant, but proportional to the time. Efflux over a spillway. If Q' is a linear function of t, that is, is proportional to t, when t is reckoned from a special origin, we have a case that is suggested by the graph of the Cane Creek flood of the author's Fig. 12; since this curve, *a-c-d*, etc., may be considered to be made up of a number of consecutive straight lines. Among the more prominent of these straight lines is the portion, *c-d*; and this portion has been treated by the writer in the attempt to discover the law connecting the variables, H and t, holding good for any instant of time between about 9.30 and 10.30 A. M.

If a straight-edge be applied to the diagram in Fig. 12, it will be found that the prolongation of the straight line, *d-c*, cuts the time-axis very closely at 9 A. M.; and that the point, *d*, corresponds to a value of $Q' = 5\,000$ cu. ft. per sec. at 10.30 A. M., that is, 5 400 sec. along the time-axis from 9 A. M. Hence, if we reckon t from 9 A. M., we have the proportion, $Q' : 5\,000 :: t : 5\,400$, or $Q' = 0.926t$ (for the foot and second as units).

Since the elevation of the surface of the water at this stage of the flow is less than 140 ft., only two of the three spillways will be in action, namely, the "old" and the "new"; and these have the same

* Detail will be found on p. 200 of Frizell's "Water Power", First Edition.

† On pp. 362 and 430 of *Engineering News*, November and December, 1901, will be found interesting matter relative to this, by the Messrs. Gould.

crest elevation of 136.0 ft. and hence are equivalent to a single spillway for which the rate of discharge, or "capacity" (see Fig. 11), is Mr. Church.

$$Q = 3.65\ (150 + 133)\ .\ H^{\frac{3}{2}};$$

that is,

$$Q = 1\ 033\ .\ H^{\frac{3}{2}} \text{ cu. ft. per sec. (with } H \text{ in feet).}$$

Again, in Fig. 10, it is to be noted that the curve of reservoir capacity is nearly straight between Elevations 134.0 and 138.0, the gain of volume in that interval of 4 ft. being 500 000 000 gal. or a fairly constant rise of 125 000 000 gal. per ft. Division by 7.58, and by 1 ft., gives 16 710 000 sq. ft.* as a fair estimate of the area, A, in the region of Elevation 136.0. These expressions and values having been inserted in Equation (1), there is obtained, after reduction and division (for foot-second units),

$$10\ 000\ 000\ \frac{d\,H}{d\,t} = 0.554\ t - 618.4\ H^{\frac{3}{2}} \qquad (6)$$

This differential equation not admitting of a strict mathematical solution,† resort was had to plotting a curve with H as ordinate and t as abscissa (t is reckoned from 9 A. M. and H is the head on the spillway), a special method being used for the purpose, giving a very close approximation.

It is a property of a circular arc that the tangent lines drawn at its extremities intersect at a point equi-distant from the points of tangency, and the method in question consists virtually in drawing, "tandem", a number of very flat circular arcs, the radius of each being the average radius of curvature for the small extent of curve involved. The arcs themselves are not drawn, being sufficiently defined by the extremities and their tangents.

For each of the five values of H: 0.6, 0.7, 0.8, 0.9, and 1.0 ft., the value of the derivative, $\frac{d\,H}{d\,t}$, was computed from Equation (6) for each of some six or eight values of t, chosen so that, by previous inspection, the true value of t for the assumed H would probably lie within the range taken; and these thirty or forty values were tabulated.

In Fig. 37, for the starting point, D (corresponding as regards time to points, b and c of the author's Fig. 12), we have $H = 0.50$ ft., and t will be taken as 2 034 sec. (that is, from 9 A. M.; see Fig. 12). Also, the value of the "slope" of the tangent line at D, namely, $\frac{d\,H}{d\,t}$, is found from Equation (6) to be $\frac{0.0910}{1\ 000}$. A straight line, $D\,F$, is then

* From the curve "area in acres" of Fig. 10, we might also have taken $A = 386$ acres $\times\ 43\ 560 = 16\ 820\ 000$ sq. ft.

† Information on this point was kindly furnished by James McMahon, Professor of Mathematics, Cornell University.

Mr. Church.

drawn through D at such an angle that a portion having 1 000 sec. as horizontal projection has 0.0910 ft. as vertical projection. Use was made of co-ordinate paper with twenty divisions to the inch, and the scales adopted were 1 000 sec. to 5 in. horizontally, and 0.10 ft. to 2.5 in. vertically.

A point, E, is then found by trial on the horizontal line, GE, that is, for $H = 0.60$ ft., such that for the value of t for this point, E, the straight line drawn through E and having a slope, $\frac{d\,H}{d\,t}$, obtained from Equation (6) for these values of H and t, intersects the previous tangent line, DF, in a point, F', making $\overline{F'D}$ equal to $\overline{F'E}$. We have thus determined very closely a new point, E, of our curve and the tangent line, $F'\,EJ$, at this point. A similar procedure, involving the tangent, $E\,J$, and the horizontal line for $H = 0.7$ ft., gives another point on the curve and also its tangent line; and so on.

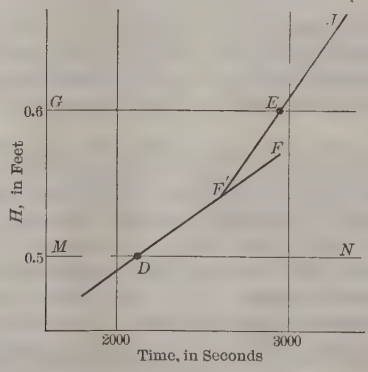

FIG. 37.

The five points thus obtained in addition to the starting point, D, are plotted in Fig. 38 and give the curve there shown. The figures annexed to the points give the abscissas (time), in seconds.

Investigations involving a change of origin and the plotting of the logarithms of the new co-ordinates, brought out the fact that the relation

$$H = 0.30 + \left(\frac{0.025}{10\,000\,000}\right)(2\,000 + t)^{2.19} \quad \ldots\ldots\ldots\ldots (7)$$

is practically the equation to this curve, within the limits shown; giving results not more than 1% in error when H is solved for. The t of this equation is reckoned from 9 A. M., as before.

Mr. Church.

For the point, *d*, in the author's Fig. 12, the value of t is 5 400 sec. and this value, in Equation (7), gives $H = 1.04$ ft. For this point the author has 137.0 ft. as elevation of the surface; that is, $H =$ just 1.00 ft.; and, under the circumstances, this may be considered a very fair agreement of the two methods.

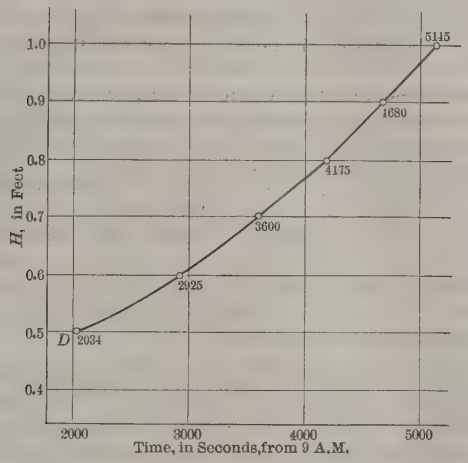

FIG. 38.

Mr. O'Shaughnessy.

M. M. O'SHAUGHNESSY,* M. AM. SOC. C. E. (by letter).—The author is to be commended for the thorough manner in which the reconstruction of this dam was undertaken. The complete way in which the details of the construction and the reasons therefor have been described are also worthy of commendation.

To a Western engineer, the extraordinary flood flow per square mile, of the region which drains into the dam, appears to be alarming. It would seem that the new spillway capacity, more than six times the original size, should surely be ample to underwrite the structure against flood damage in the future. It would also seem that this hollow, reinforced concrete type of dam was well adapted for this particular site, and that its original failure was due, not to the type of structure, but, in a large way, to lack of adequate care with the foundation portion of the work. Neglect of this nature is likely to cripple any dam, and one of this type perhaps less than any other, as a fracture of a couple of panels will release the stored waters and safeguard the rest of the structure.

* San Francisco, Cal.

Mr. O'Shaughnessy.

One of the cardinal principles to be observed in any dam foundation is to remove the uplift pressure and make a perfect contact between the water-tight skin or face of the dam and the solid reliable foundation material. The results of tests on the friction resistance of clay soil and the analysis of the sliding factors furnish data of original labor, which are creditable.

The method of pressure grouting the footings is a matter of interest, and apparently this work was very well done in the reconstruction.

The observation that "great care is necessary even in foundation work, where it is too often assumed that any kind of work will suffice", should be impressed on the minds of all young engineers as the most important of all the elements that need attention. No matter how perfect the superstructure, if the foundation is inadequate, final failure or consequent trouble will result. If the leakage on May 25th, and November 1st, 1915, was actually measured, the facts would be of interest.

The reason for the construction of a curtain-wall and roof to avert the freezing of the top of the weep-holes and deep drains, is a matter of much interest to engineers working in a country which is practically frostless.

The author's comments on cracks in the original structure from lack of proper contraction joints is illuminating. The writer's experience has been that where large masses of concrete were exposed to rapid temperature changes, there should be joints at least 40 ft. apart, and, in ditch lining, he has used joints as close as 12 ft. to advantage.

With reference to asphalt in expansion joints, there is a tendency of this material to flow in hot weather. Such a condition was studied in the reconstruction of the Twin Peaks Reservoir, in 1912, and by adding a percentage (17½% by weight) of diatomaceous, kieselguhr, or infusorial (different names for the same substance, which is almost pure silica), to the asphalt, this tendency to flow was removed, and the asphalt still retained its plastic condition.

In stating the cost per cubic yard, the details of the cement and iron costs are missing, and would be interesting if added by the author.

The writer has seen no paper, for some time, which has been so candid in every manner in describing all the engineering features, so that brother engineers can profit by the experiments and experience gained in this structure.

Mr. Justin.

Joel D. Justin,* M. Am. Soc. C. E. (by letter).—This paper teaches a much needed lesson; a lesson not so much needed by the Engineering Profession as by the promoters and owners of projects in connection with which the construction of dams is a feature.

* Chippewa Falls, Wis.

Mr. Justin.

If an owner is considering the advisability of constructing a dam, there is one way of no regrets for him to go about the inception of such a project. If he is not already in contact with engineers versed in that particular line of work, he can readily meet them through other owners who have built dams. Having chosen his consulting engineer, the owner should authorize him to make a complete preliminary report on the project, including the economic features. The extent of such preliminary report and investigation should be left largely to the judgment of the engineer, just as the number of visits that a doctor makes his patient should be left to the judgment of the doctor, and not to that of the patient.

Of course, such an engineer should have nothing to sell except service. If the preliminary report is favorable, this ideal owner will say to the engineer:

"Very well, then, we will build this structure as you recommend, and you will consider yourself retained to design the dam and appurtenant work, and to supervise the construction. We will expect you to have on the ground at all times competent assistants to see that your orders are executed. In this matter, Mr. Engineer, we put ourselves entirely in your hands."

It is true that the best of engineers sometimes make mistakes. So do doctors and lawyers. The difference is that the two last named professions have an excellent opportunity of "getting away with it", whereas the mistakes of the engineer stare the whole world in the face, for they can be seen and photographed. It is probably for this reason that the business man, who accepts as infallible the advice of his lawyer or doctor, questions and often ignores the advice of his engineer. It is certain, however, that if a larger proportion of owners were to follow the course of action just outlined, there would be fewer failures charged to the Engineering Profession.

In the case of the Stony River Dam, a competent consulting engineer was retained, apparently merely to investigate the feasibility of the site. His connection with the project ceased when the contract for the construction was let, and he apparently had nothing to do with the design. Had he also been retained to design and supervise the construction of the dam, it is improbable that Mr. Scheidenhelm would now be writing about its reconstruction.

Hollow dams of the type used at Stony River have manifest advantages over other types for the conditions existing at the chosen site. In many such dams insufficient attention has been given to foundation problems. It would almost appear that it has been quite customary to use what may be designated as stock designs, without much regard for local conditions, and without very thorough sub-surface investigations.

Mr. Justin.

The paper proves that the author went into this matter in a most thorough and complete manner. He realized that a reconstruction of this nature required as much if not more engineering investigation than an entirely new project. In spite of his convincing hydrological study, Mr. Scheidenhelm's provision for a spillway capacity of 1 840 sec-ft. per sq. mile seems rather high. Much of this, it is true, is obtained by merely allowing spilling over the bulkhead section or "Intermediate Spillway." It appears probable, from the author's study, that the duration of a flood when the capacity of the original spillway would be exceeded would be brief. Consequently, it would seem that the author might have saved the cost of the down-stream apron on the new spillway. The abutments at the east and west ends of the masonry structure might have been raised several feet to prevent water flowing over the earth, and then, when the capacity of the old spillway section was exceeded, discharge would take place over practically the entire width of the dam. It would have been necessary, of course, to extend the tumbling hearth for the entire distance. It would probably also have been desirable to provide an additional apron to protect the soil just down stream from the dam.

The author shows most conclusively that the resistance to sliding of the structure as originally built was insufficient. This condition is no doubt quite often the case with existing hollow reinforced concrete dams built on soil foundations. In the design of such structures there has often been far too little attention given to making adequate provision against sliding. The methods adopted by the author for increasing the resistance to sliding are most ingenious and effective. At the same time their execution must have been expensive.

After investigation the author decided that the safe soil pressure had already been reached, and therefore he did not consider it safe to obtain the desired additional resistance to sliding by filling in with earth on the floor of the dam. This objection might have been overcome by extending the reinforced concrete floor or footings down stream from the buttresses until the desired maximum soil pressure was attained. To take the reaction from the soil pressure it would have been necessary to cantilever the buttresses out in a down-stream direction over the additional floor or apron. This cantilever would probably take the form of a triangular addition to the buttresses at their down-stream ends. The earth filling, to produce the necessary weight, would be placed inside the dam, and the bulk of it as far up stream as possible, thus obtaining a lower maximum soil pressure per unit of area at the toe than if the fill were evenly distributed.

The two problems of securing proper resistance to sliding and adequate cut-off should be handled as one, and this the author has

done. The construction of the cantilever heel, the underpinning, and the extension of the original cut-off were clearly very expensive. This work, aside from the mere grouting of leaky joints and of voids along the cut-off, would be omitted in the method the writer has in mind. In the case of a dam on a soil foundation, the construction of an impervious cut-off to rock is frequently out of the question. The problem is more generally one of introducing sufficient frictional resistance to the flow of water under the dam, so that the velocity of the water issuing from the down-stream toe is insufficient to move the foundation soil. In fact, the desirability, in the case of a yielding soil, of a deep cut-off at the heel to rock is open to question, as it produces a tendency to increase the maximum soil pressure at the toe. The necessary frictional resistance to the flow of water under the

Mr. Justin.

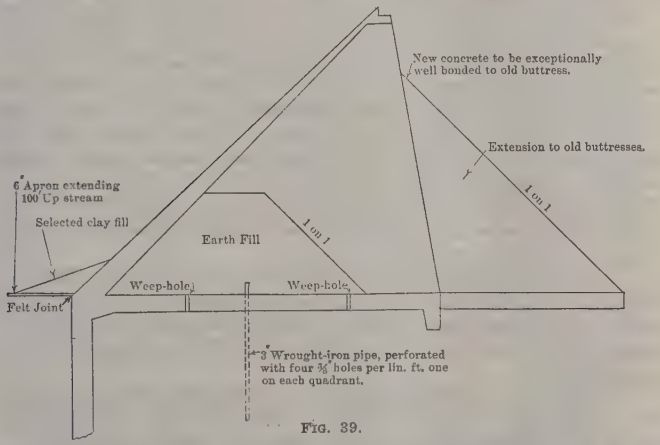
New concrete to be exceptionally well bonded to old buttress.
Extension to old buttresses.
1 on 1
6" Apron extending 100' Up stream
Selected clay fill
Earth Fill
1 on 1
Weep-hole
Weep-hole
Felt Joint
3" Wrought-iron pipe, perforated with four 3/8" holes per lin. ft. one on each quadrant.

FIG. 39.

dam is obtained by increasing the distance through which the water must flow before it can escape at the down-stream toe. This may be obtained by (1) increasing the depth of the cut-off, (2) by increasing the width of the base, (3) by a combination of these two methods.

The additional down-stream floor or apron, suggested by the writer as a method of obtaining decreased soil pressure, would, by increasing the frictional resistance, decrease the velocity of the water flowing through the soil. This, however, would probably not be sufficient. Therefore, the writer would use an up-stream apron. The thickness of this layer need be only 6 in. or indeed it might consist of a few layers of "Gunite" on a light-mesh reinforcement, there being slight stresses to be resisted. For the typical section, the width of this apron should probably be about 100 ft., as it would not be wise to

Mr. Justin. place great dependence on the original cut-off. At the junction of the apron and the heel there should be a special flexible joint which would remain tight even if there should be a slight movement in the dam. A small fill of selected clay should also be placed at this point. The alternative for providing against seepage and sliding here suggested is shown in Fig. 39, the dimensions being merely illustrative.

In outlining this method of increasing the resistance to sliding and to seepage, the writer realizes that it was doubtless considered by Mr. Scheidenhelm. The only reason for suggesting it is, that, from the author's description, it would appear that it would have been cheaper than the equally efficacious one adopted.

Mr. Riegel. Ross M. Riegel,* Assoc. M. Am. Soc. C. E. (by letter).—The writer desires to commend the practice of using a hypothetical storm with a graph of the run-off, in determining spillway characteristics. His observation has been that, in the design of most dams, this determination has been largely a matter of conjecture, as is attested by numerous failures. The author's practice is similar to that of the Miami Conservancy District in the design of flood-control dams for the Miami Valley in Ohio. In the latter case, recorded gauge heights enabled a graph of the run-off to be constructed for the great flood of March, 1913. From this, by the addition of a margin of safety of about 40%, another graph was developed for the determination of the spillway and outlet capacities for the dams.

Although the spillway capacity of the Stony River Dam is high, as compared with the usual practice, the circumstances of high altitude, heavy rainfall, and small drainage area, and the fact that the dam as reconstructed must not fail, combine to justify an attitude of great conservatism.

The essentials of dam construction, aside from spillway capacity, are a tight face to retain the water, a stable structure to maintain the integrity of the face, and satisfactory drainage facilities to carry away any water which may pass the face or the foundation. In all these essentials the original dam appears to have been deficient. The tight face was wanting by reason of an insufficient and faulty cut-off; the drainage provision was such that it was subject to interference by freezing during a great part of the year; and the resistance to sliding seems to have been markedly deficient.

In the reconstruction, these faults have been remedied with a degree of thoroughness not often seen. The improvements at the heel, as well as the various extensions and underpinning of the cut-off, have corrected the faults in the original face. In his discussion, Mr. Rutenberg seems to have overlooked this point entirely. Had nothing been done to improve the cut-off, the drainage provisions of the recon-

* Dayton, Ohio.

structed dam might indeed have been insufficient, and Mr. Rutenberg's points would have been well taken. The fact that such leakage as did occur after the dam was refilled was readily checked, or checked itself after a short time, is ample testimony to the effectiveness of the measures taken. Mr. Riegel.

The provision made to protect the drainage outlets from freezing, namely, housing in the base of the dam, savors of the heroic, but seems to be justifiable. The author was repairing an existing structure, and to have developed an ideal system of sub-drainage with protected outlets below the dam was evidently prohibitive as to cost. Under the system adopted, any seepage water will find its readiest avenue of escape through the weep-holes or pipes. Its escape immediately under the footing will be practically prevented by the new "toe" wall. Moreover, after passing the improved cut-off, the motion of seepage water through any strata below this wall will be so slow that no erosion need be feared.

The provision of increased resistance against sliding seems to have been necessary and to have been well worked out. The writer fails to see any better method of remedying this defect, and feels that the device is worthy of imitation. Although the criticism has been made that the heel- and toe-walls are too light to carry the entire horizontal thrust, it seems to the writer that such a requirement would be excessive. Their function is to add to the resistance normally offered along the top surface of the foundation itself, and, therefore, their strength is doubtless sufficient.

On the whole, the writer has been impressed by the care and thoroughness of the author's methods. He wholly fails to comprehend Mr. Rutenberg's criticism, which seems to have been based on rather hasty consideration. For instance, his remark that the dam is less safe now than before reconstruction seems particularly unsound, inasmuch as the original dam failed after being in ordinary service for 65 days, and the reconstructed one has now been in service for 2 years.

F. W. Scheidenhelm,* M. Am. Soc. C. E. (by letter).—The discussion of the paper has been very interesting to the writer, as presumably it has also been to other engineers concerned with the problems treated. Among other things, it serves to emphasize the fact that there is rarely agreement among engineers as to the exact measure, or margin, of safety which should be given to, or is possessed by, a structure. On the whole, the writer finds much cause for satisfaction in the comments contained in the discussions. Mr. Scheidenhelm.

Frequently, papers presented before the Society and the discussions thereon result in a symposium of data and opinion on certain subjects. Such is the case with regard, in particular, to two problems which were involved in the reconstruction of the Stony River Dam, and are treated in the paper.

* New York City.

Mr. Scheidenhelm.

The first of these problems concerns the determination of the maximum flood for which spillway capacity is to be provided, and the consequent determination and design of the spillway provision itself. Of the ten members who have discussed this matter, only one, Mr. Gregory, feels that the writer has "surely gone beyond the necessary limit." The great majority, however, apparently agree on the proposition that a drainage area as small as that of Stony River at the dam site requires entirely different treatment than do larger areas, and that the exceptionally large maximum run-off adopted, although surprising, is warranted.

Referring to the comment of Mr. Gregory (page 1060) the writer feels that it is certainly necessary to take into account, not merely rainfalls of 2 or 3 hours' duration, but those of considerably longer periods, and more especially their consequent floods. This is for the reason that from such rainfalls the run-off (into the reservoir) which immediately precedes the maximum or critical periods of, say, 2 or 3 hours, necessarily affects the absorption or equalization capacity of the reservoir during the vital period.

It is true, as pointed out in the paper, that a record or graph of run-off never truly represents the corresponding rainfall; that is, the maximum rate of run-off is always less than the maximum intensity of rainfall. Yet, even making full allowance for this fact, as applied to actual Stony River conditions, it is reasonably evident, from the studies depicted in Figs. 12 and 13, that the length of the period of rainfall which should be considered is at least 6 hours; and it must be remembered that rainfall does not persist at a uniform intensity through any 24-hour period. Thus, were there an automatic gauge record available for the 22-in. rainfall during 24 hours at Altapass, N. C., on June 15th-16th, 1916 (page 930), the record for the maximum 6 hours would probably be astounding.

Again, looking at the matter from Mr. Gregory's point of view, and making the reasonable assumption, size and topography of drainage area considered, that "the peak of the run-off at the dam is equal to the greatest average rate of rainfall during any 45-min. period" (page 940), it is to be noted from Fig. 15 that "the most severe run-off conditions reasonably conceivable", there shown, indicate a rainfall with an intensity averaging only about 4 in. per hour for the maximum 45 min. This intensity is somewhat less than that given for the same period by the Talbot formula, quoted by Mr. Gregory, and, of course, is less, by an even greater amount, than the corresponding "Maximum for Greater New York" (Fig. 32). Incidentally, the Talbot and the Greater New York curves of Fig. 32, when plotted on Fig. 9, fall outside of the group of curves shown thereon; in other words, the curves of Fig. 32 show the greater intensities of rainfall.

Mr. Scheidenhelm.

It is interesting to note that apparently the Cane Creek and Elkhorn Creek floods, which formed the basis for the studies presented in the paper (page 926) were of such intensity as not to be included within the formula proposed by Weston E. Fuller, M. Am. Soc. C. E.* The same is true also of a run-off of 1 302 cu. ft. per sq. mile, reported† for the drainage area of 143 sq. miles of Devil's Creek, near Viele, Iowa.

The method of determining or checking spillway capacity by means of graphs of hypothetical maximum floods to be provided against, seems to meet with approval. Similar methods are shown to have been used in the design work of the Miami Conservancy District at Dayton, Ohio, as referred to by Mr. Riegel (page 1086), and discussed in considerable detail by Mr. Grant (page 1067). A remarkably close agreement of results is evident from comparison of Figs. 12 and 36, the former being based on the method used by the writer, whereas the latter is based on the flood routing method applied on the work of the Miami Conservancy District.

The second feature provoking considerable discussion is the utilization of the underlying foundation soil in setting up resistance to sliding. On this subject the discussion of Professor Cain is a contribution of exceptional value.

One may fairly question whether present experimental information regarding the behavior of soils is sufficient to warrant the acceptance of Professor Cain's application of Coulomb's laws to cases, such as that of the Stony River Dam, where there is involved the question of sliding, and more particularly of impending sliding. In fact, the writer still feels constrained to distinguish between the conditions existing before a body of clay has failed in shear along a given plane or surface and the conditions existing after the resistance to shear has been overcome (that is, when, in his terminology, cohesion has given way to friction, plus any adhesion). Such distinction does not appear to be inconsistent with Coulomb's laws, especially as expressed by Professor Cain somewhat more broadly in his paper on "Cohesion in Earth."‡ In fact, Professor Cain's more frequent use of the term "shear" in his analyses based on Coulomb's laws, as applied in his discussion (page 1045), is significant. Even Professor Cain now considers that possibly not all "cohesion" (Cain) is destroyed on the beginning of the actual motion or sliding.§

However that may be, all must be in full agreement with Professor Cain in his emphasis of the "need for comprehensive experimentation

* Flood Flows," *Transactions*, Am. Soc. C. E., Vol. LXXVI, p. 564.

† *Water Supply Paper No. 162*, U. S. Geological Survey.

‡ *Transactions*, Am. Soc. C. E., Vol. LXXX, p. 1316.

§ This appreciable part of "cohesion" everted in addition to friction during the actual sliding of clay on clay (p. 1045), the writer would term "adhesion", as more accurately describing its nature.

Mr. Scheidenhelm.

to determine the coefficients of cohesion." Moreover, the writer believes that Professor Cain has performed an important service in presenting in such detail the several analyses of the Stony River problem on the basis of coherent earth. It is not improbable that, as the result of such work as is now being carried on by the Society's Special Committee on Soils, the treatment advanced by Professor Cain may be adopted, at least in part, and especially as applied to cases of actual sliding.

The writer agrees entirely with Professor Cain that probably the effect of a longer time of application of a given shearing load on a body of clay under test would be to afford lower values of unit shearing resistance (page 1046). It should be noted, however, that, in the Stony River design, no reliance was placed on the shearing resistance of the foundation soil.

In the interesting analyses (pages 1046-48) of the original structure before failure, based on the theory of coherent earth, Professor Cain unwittingly made several erroneous assumptions.

The mistake apparently was caused by his treatment of the section of dam shown on Plate XII as being taken at the maximum height portion of the dam. This is not the case, for Plate XII shows the "Typical Section at Bay 35" referred to in Columns (3), (5), (7), and (9) of Table 3. The resulting error in the value of Q, as derived by Professor Cain (page 1047), is not serious, but, unfortunately, the same mistake made later in his discussion robs the results of their significance, though, of course, it does not deprive the theoretical analysis of its value as such.

Thus, in assuming anchoring walls applied to the original structure, as shown on Plate XII, Bay 35, Professor Cain perforce used loads (page 1048) as given in the writer's Cases I and II (pages 956-958) and applying to the sections of maximum height, instead of to the section at Bay 35. The difference in height of sections will be apparent on reference to Plate IX. Moreover, the loads used by Professor Cain apply to the structure in its original condition, whereas both concrete and water weights were greatly increased in the reconstruction, without any increase in the horizontal water thrust (Table 3). The horizontal water thrust for Case I, for instance, is but 53 820 lb. per lin. ft., instead of 81 000 lb., as assumed by Professor Cain.

In consequence, the conclusions as to resistance to sliding, furnished under the respective hypotheses set up under the theory of coherent earth, are not applicable. The corrected figures show much larger margins of safety. Nevertheless, it appears to the writer that the appropriateness of the methods used by Professor Cain has in no wise been affected; and, for the purpose of theoretical study, those interested may well disregard the inapplicability of the figures.

Mr. Scheidenhelm.

Professor Cain neglects the resistance along *BC* and *IJ* of his Fig. 28, on the ground that there is "but little weight over parts of these surfaces" (page 1053). It should be evident, however, on study of Plate XII and the description of the toe-wall and its effect (page 964), that the new toe-wall in essence extends the base of the dam as far as *F* (Fig. 28), and that, in fact, the greatest vertical loads on the soil occur at the very down-stream edge of the toe-wall, namely, just to the left of the plane represented by the line, *FC*. For this reason the writer believes that one is not warranted in omitting from consideration the resistance along *BC* and *IJ*.

The graphical solution for the passive resistance of the down-stream foundation soil is exceedingly interesting, and here again those interested incur a debt to Professor Cain. The point made by him, that the writer was too conservative in assuming zero load at the free surface of the earth at *F* (Fig. 28), is well taken, as is evident from the analyses (pages 1057-58).

On the subject of the safety of the structure, as repaired and strengthened, the measures taken appear to have general approval, except by Mr. Rutenberg. In view of the nature of the discussion submitted by him (pages 1029-35), the writer is loath to make a detailed reply. However, a number of the premises adopted by him diverge so widely from the facts that it appears advisable to examine some of the points which he has raised, in order properly to co-ordinate the paper and the discussion thereof. Except in special instances, the writer will refrain from discussing mere expressions of opinion appearing in Mr. Rutenberg's contribution, for, of course, each engineer is entitled to his own opinion, and it would be futile to attempt to obtain agreement on all points:

1.—With reference to the discussion on remedying the faulty condition of the foundation soil (page 1030), the writer desires to emphasize that reliance against leakage was placed solely on the extension and perfecting of the cut-off wall, and not on the compacting of the foundation soil by grouting. The object of the grouting, as stated, it is believed clearly, on pages 987-988, was to compact the soil for bearing purposes. In other words, it was to remedy conditions caused by leakage which occurred prior to the failure of the dam; or, in the words of the desideratum expressed by Mr. Rutenberg himself, "to fill, if possible, all the holes found in the foundation soil" (page 1033). It will be noted that the subject of pressure grouting was treated by the writer primarily under the heading of "Footings", and not under that of "Leakage and Drainage."

In passing, it may not be out of place to remark that it is the writer's experience that pressure grouting in porous soils and gravels is not a sure means of causing water-tightness, but that, on the other hand, such grouting is generally effective in rock having pronounced seams.

Mr. Scheidenheim.

2.—The writer agrees in essence with Mr. Rutenberg's statements in the last paragraph on page 1030, but cannot agree with his conclusion as expressed in the paragraph following. The writer must still insist that "the smaller the coefficient of frictional resistance, the less the net effect of uplift pressure." Perhaps this anomaly of agreement on certain premises and disagreement as to the conclusion is explicable on the ground that Mr. Rutenberg has not noted that the writer's statement applies solely to the effect of uplift pressure in so far as resistance to sliding is concerned. This is manifest from the context (page 947).

The meaning of the writer's statement can perhaps be made more clear by using a numerical example. Thus, if over a given area of base of dam there should exist an uplift pressure of 100 000 lb., there would result the equivalent of a net reduction in weight by the same amount. The net reduction in weight, however, is a different thing from reduction in resistance to sliding. For instance, if in this hypothetical case the coefficient of frictional resistance were 0.75, the resistance to sliding would, by reason of the assumed uplift, be decreased by 75 000 lb. In the case of the Stony River foundation soil, however, the coefficient of frictional resistance probably averages about 0.33. This coefficient, applied to the uplift condition just assumed, would result in decreasing the resistance to sliding by only 33 000 lb.

3.—The expression of opinion that leakage and percolation in the dam should have been intercepted "directly and immediately on the up-stream face of its cut-off wall, and discharged down stream by special channels without any pressure" (page 1031) is so unique and, in the writer's opinion, illogical, that he hesitates to believe that it was intended to make such a statement. The writer insists that one need be concerned only with such water (leakage) as passes through the diaphragm (intended to be water-tight) of which the cut-off wall is a part. Of course, the primary effort was devoted to making the cut-off wall as tight as possible. In this regard, the results of the reconstruction speak for themselves, namely, that at present the merest trickles of water pass through or under those portions of the dam founded on earth over-burden.

4.—Although desiring to avoid useless quibbling over terms, it seems necessary to point out that the statement imputed to the writer, "that water under pressure in the foundation soil is never harmless" (page 1031), is not warranted by the statement that, in the case of the Stony River foundation, deep-lying pockets, if any, of water under pressure are "relatively harmless" (page 997). By "deep-lying pockets of water" the writer meant accumulations of water, under pressure, existing at elevations below the bottom of the new anchoring wall at the heel; in other words, below the probable location of the main "planes of least resistance." Such deep-lying pockets of water are relatively

Mr. Scheidenhelm.

harmless, partly because of their depth, and partly because of the comparatively small coefficient of frictional resistance obtaining in the Stony River overburden, the effect and relation of which coefficient to uplift pressure and resistance to sliding have been explained in Paragraph 2.

5.—The evidently unintentional misstatement is made that the writer "admits the presence of considerable hydrostatic pressures at a distance of 20 ft. from the down-stream side of the cut-off wall" (page 1031). The writer's actual statement, as given on page 998, is believed to be self-explanatory to a careful reader. The writer does not admit the presence of such considerable hydrostatic pressures; on the contrary, he desires again to point out that no pressure head of more than 2 ft. has been detected at the top of any weep-holes or drain pipes since the reconstruction was completed (pages 999-1000).

6.—The premises being erroneous, as shown above, Mr. Rutenberg arrives at a conclusion manifestly erroneous, namely, that "the up-stream half of the base is exposed to dangerous uplift pressures" (page 1031). This erroneous conclusion leads to a likewise erroneously applied train of "previously stated consequences" (for which see page 1031).

7.—Again, there is imputed to the writer cognizance of the actual existence of the rather strained hypothetical phenomena which Mr. Rutenberg sets up as to the possibility that "the drainage pipes will be filled through the higher perforations from the higher strata of the foundation soil, etc." (page 1032). It is conceded that such conditions might exist, but, if so, it is far from certain that the results would be harmful. Be that as it may, however, the phenomenon referred to by the writer (page 1000) clearly lends no weight whatsoever to the hypothetical conditions proposed.

Relative to this matter, it should be noted that the fact that so many of the weep-holes and drain pipes remain just full of water, but not over-flowing, is sufficient proof that the source of such water is ground-water, and that it is not the result of leakage from the reservoir. The writer is inclining more and more to a belief in the correctness of the explanation of the phenomenon on the basis of capillarity.

8.—From a casual inspection of Plate XII, Mr. Rutenberg ventures to pass judgment as to the strength of the anchoring wall at the heel. That the inspection must have been casual is shown by the fact that, contrary to his statement (page 1032), Plate XII does give the essential dimensions, and even the details, of the reinforcement of the anchoring wall at Bay 35. Furthermore, data as to the anchoring walls at Bays 30 and 31 are shown on Plate XIII.

Mr. Scheidenhelm.

An exposition of the detailed calculations involved in the design of the anchoring walls might have been included in the paper. However, even as it stands, the paper has been characterized as exhaustive. The addition of such an exposition, therefore, did not seem advisable, and at this date the military duties of the writer preclude such diversion on his part. Hence it must suffice to state that the anchoring walls at heel and toe were designed, with standard factors of safety, so as to carry, jointly and safely, all the horizontal thrust, excepting only an amount equal to the product of the weight of the superstructure (plus water load) multiplied, in the case of the clayey soil, by a coefficient of frictional resistance of 0.2 (page 978). The latter amount of horizontal thrust can certainly be counted on to be transferred into the foundation soil directly by the frictional resistance existing between the soil and the base of the superstructure. Essentially the same method has been used by Professor Cain, in the admirable analysis contained in his discussion (page 1050-51). The only difference is that Professor Cain there considers the writer's "frictional resistance" as subdivided into friction and "cohesion."

As a matter of fact, the results obtained by Professor Cain for the load carried directly from the base of the dam into the earth, and thence down to the "planes of least resistance", without being transferred through the anchoring walls at the heel or toe, are very much greater than those obtained by the writer by the use of the coefficient, 0.2. It will be noted that the coefficient assumed by the writer for this particular purpose is only 80% of the "probable minimum value" shown in Table 1; and, as such, it is believed to be thoroughly conservative, with the purpose and result that the anchoring walls were correspondingly built so much the stronger.

9.—The fear is expressed that the toe-wall "will aid in collecting the leakage under the dam" (page 1033). Inasmuch as weep-holes were left through the footings (see Plate XII), it is not believed that the hydrostatic pressure in the foundation soil at the toe-wall can be sufficient (the depth of the wall considered) to require relief through the toe-wall. An exception exists in the case of the old spillway, where the toe-wall is especially deep (17 ft.). There, drainage openings were provided through the vertical member of the toe-wall, the details of which are shown in Fig. 27.

10.—With reference to the effect of the heel-wall and its "under-cut slope" (page 1033), the attention of Mr. Rutenberg is respectfully called to the physical fact that the heel-wall cannot possibly present additional surface for the application of uplift pressure without at the same time presenting additional surface for the application of downward water pressure, which, of course, resists overturning. Even granting that the up-stream portion of the heel-wall is subjected to full hydrostatic pressure on the "under-cut slope", the net effect of

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the heel-wall would still be to resist overturning, for the reason that the specific gravity of the heel-wall concrete is much greater than that of the water which it displaces. In this respect the location of the heel-wall so far up stream, and its correspondingly great lever arm, are of no small consequence in actually increasing the stability of the structure against overturning. Manifestly, the result is diametrically opposite to that stated in the discussion.

11.—The writer quite agrees with the implication regarding the tests of frictional resistance of various Stony River soils, to the extent that it would be dangerous to rely on these as giving truly representative results (page 1033). However, it by no means follows that the tests are "without practical value." On what is our knowledge of the frictional resistance of soils based, if not on observation and tests, as well as on theory? It should have been unnecessary again to point out what was evidently overlooked, namely, that the writer discounted the test results to such a degree that, instead of assuming an average coefficient of 0.61, he based the design on a coefficient of 0.33 (page 953). Moreover, the structure is shown actually to be safe with a coefficient of 0.25.

12.—Similarly, it should not have been necessary to remind any one that no pretense has been made by the writer that the weight of the underlying foundation soil aids, in the least, to resist overturning. The weight of the underlying foundation soil is utilized solely to resist failure by sliding. Of course, once such soil has been washed away, it can no longer resist sliding; but failure resulting from such conditions would have been due primarily to leakage or undermining; and against failure of that sort entirely different precautions have been taken.

13.—Mr. Rutenberg is under a misapprehension as to the writer's "adding 3½ ft. of water to the original level in the reservoir" (page 1034). As a matter of fact, the writer did not add to the water level, but merely made provision for taking care of floods, which would have come with equal probability in the cases of the original and the strengthened structure. True, a parapet was added to prevent overflow at the bulkhead sections and consequent undermining at the down-stream edges of those sections. However, that the additional horizontal water pressure caused thereby is negligible should be patent to every one with a moment's computation. On the other hand, the new spillway serves to hold down the water level in the reservoir for a given flood discharge, as compared with the water level which would have existed in the case of the original structure for the same flood discharge. It is pertinent to note that, for instance, even on the basis of a run-off of 1 386 sec-ft. per sq. mile (as reported for Cane Creek, page 926), the bulkhead section of the original structure would have been overtopped by 2½ ft. That is, such a discharge could have

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taken place only after the water level had risen to an elevation of about 141.5. The corresponding water level under the new conditions for a spillway discharge of 1 386 sec-ft. per sq. mile will be approximately at Elevation 141.65. Evidently, the difference is negligible.

Again, reference to Table 3 will show that the same elevation of head-water, namely 140.5, was used for the stability analyses for both the original and the strengthened structures under "normal maximum load."

14.—It is true that the superstructure of the new spillway section is heavier than that of the original portions of the dam. However, it does not follow, by any means, that either "the original structure is insufficient or the excess of the new is useless" (page 1034). As to the original structure, the writer was forced to choose, on the one hand, between recommending the expenditure of a relatively enormous sum of money in order to strengthen the decks and buttresses, and, on the other hand, accepting somewhat higher working stresses than would have been allowed had the structure been designed initially for the water loading for which it should have been designed. As to the new spillway, the increased cost of using normal working stresses, as compared with the cost for the slightly higher stresses obtaining in the original portions of the dam, for the same loading, was not great, and there would have been no justification for holding down the new spillway dimensions to correspond with those of the original structure (page 1005). A situation of this sort requires the use of common sense as well as the application of theory.

15.—The unit bearing stresses for the soil and the base of the dam, the necessary data for the computation of which Mr. Rutenberg misses (page 1034), are set forth in considerable detail on pages 978-982. Further details might have been added had it been desirable to extend indefinitely the limits of the paper.

16.—The writer is unable to agree that the reservoir or ground-water leakage, mentioned on page 1000 and discussed on page 1035, has found other opportunities for "harmful erosion." Mere differences of opinion are immaterial, but it is to the point to know that since about a month after the reservoir had completely filled, there have been no further appearances of muddy water at any of the many drainage openings in the dam. This fact would seem to bear out the writer's opinion (page 1000) that the muddy nature of the water which appeared was due to adjustments in the foundation soil to the new conditions of loading.

The foregoing comments have been essentially on divergences from the facts. However, certain mere expressions of opinion on the part of Mr. Rutenberg seem to warrant passing mention. For instance, he "believes the reconstructed dam is less safe than the original structure" (page 1035) and "is firmly convinced that the reconstructed dam will

also be broken, perhaps soon, if adequate repairs are not made in time" (page 1035). It is unfortunate that opinions of so grave a nature and predictions so dire should be advanced, based, as they are, on erroneous premises, and certainly not substantiated by the facts. Probably the most important considerations in this regard are the results of the reconstruction, which speak for themselves:

a.—In the first place, the reconstructed dam has now stood for more than 2 years under full water load. The original structure stood only 65 days under full water load prior to failure.

b.—Shortly after the reconstruction was completed, the dam was subjected to a flood which caused the head-water to rise about 3½ ft. higher than the highest water level which had occurred during the short life of the original structure.

c.—Finally, an inspection made recently by the writer showed that the dam is in excellent condition.

Certain criticisms have been offered by Mr. Moore (page 1035), Two of these in effect duplicate criticisms made by Mr. Rutenberg, namely, as to the value of the writer's tests of frictional resistance and as to the structural strength of the new anchoring walls at the heel of the dam. The writer's comments in regard to these criticisms may be found in the paragraphs numbered 11 and 8, respectively.

The contingency of the fracture of the anchoring wall at the heel, to which Mr. Moore refers, is not believed by the writer to be a serious one. The extent to which such considerations are pertinent is a matter of judgment, as to which it is not surprising to find differences of opinion. Had it been considered necessary by the writer, the heel-wall and toe-wall could, and would, have been made even deeper, thus further reducing the unit soil pressures caused by such horizontal thrust as is transferred by the anchoring walls into the foundation soil. The writer desires, further, to point out that it would be impossible to have uplift pressures from both clay and water acting on the same areas of the base of the dam. Inasmuch, therefore, as certain allowances were made for the uplift pressure of water (Table 3), it is evident that considerable provision has already been made for flotation effect, if any, on part of the foundation soil.

The "floating off" of a dam is, in the writer's opinion, a fanciful expression rather than a serious possibility. He has yet to learn of an actual case of floating of a dam on either water or soil. Nevertheless, he does not mean to belittle the possibilities and effect of uplift pressure, as is shown by the load assumptions set forth in Table 3 under the "most severe conditions within limits of reason."

Again, Mr. Moore states that the water assumed to flow over the intermediate spillway would not fall "well-nigh" vertically. Although desirous of avoiding any hair-splitting discussion as to what "well-nigh" vertically means, the writer wishes to state that the upper and

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lower nappes of the over-falling water were determined according to the results of the experimentation of M. Bazin, and the tumbling hearth mentioned (page 931) was designed accordingly. It is interesting to note that Mr. Brodie in his discussion (page 1043) mentions that his own checking gave results practically identical with those of the writer for the shape of the new spillway crest. The same method was used by the writer in determining the shape of the new spillway crest as in determining the nappes of the over-falling sheet of water at the intermediate spillway. The fact that both studies were based on the work of M. Bazin explains the close check between the results obtained by Mr. Brodie and by the writer.

The real value of the use of the brittle steel pins as flash-board supports is no longer a matter of doubt; the results of three seasons of operation speak for themselves. Thus far, the pins and flash-boards have allowed the undisturbed and safe utilization of the 25% additional capacity of the reservoir made available by the use of the flash-boards. The writer does not pretend that any and all brittle steel pins will accomplish such results, but does believe that merit is justly attributable to the pins adopted in the present case.

In addition to the foregoing, certain scattered comments appear to be appropriate to round out the closure of the discussion:

Mr. Finch very properly emphasizes the advisability of paying more attention to the architectural treatment of dams, etc. (page 1029). In this the writer entirely agrees, even though special treatment was not necessary or warranted in an isolated location like that of the dam under discussion. The type of treatment, of course, had been fixed before the writer's arrival on the scene. It is noteworthy, however, that the construction of a second spillway gave a balancing effect, especially to the down-stream view of the dam, which the original structure entirely lacked.

With reference to Mr. Downs' discussion of the coefficients of frictional resistance of Appalachian materials (page 1036), it is well to remember that though the tests showed an average coefficient of 0.61, the designs were actually based on a probable coefficient of 0.33, and that it is designed to be safe for a coefficient of 0.25, without reliance on the shearing resistance of the clayey soil.

On the several questions raised by Mr. Dunham (page 1039), the writer is unable to shed any light. Most of the questions do not fall within the scope of his investigation, and it is not believed to be appropriate to extend the present discussion by a consideration of the possibilities of imitating Nature's regulation of the flow of streams. The use of the increased flow of the river for power purposes was not contemplated.

Mr. Gregory expresses concern as to the advisability of placing tar-paper in the joints of a dam (page 1060). In the case of this dam,

this material was not expected to be permanent, and it is believed that, as the tar-paper gradually disappears, in years to come, it will simultaneously be replaced by sediment and accretions resulting from the action of the reservoir water on the concrete, particularly the alkaline elements thereof. Mr. Scheidenhelm.

It was economy that dictated the use of such an expedient as tar-paper; yet its immediate effectiveness appears to have been as great as one could expect from any other method. The leakage through the joints has actually become negligible, and that, of course, is the result sought. A leakage proportionate to that reported by Mr. Gregory as having existed at the Ashokan Dam (page 1064) would have been intolerable at Stony River, in view of the relatively small reservoir capacity available in the latter case. It is interesting to note that the initial daily leakage at the Ashokan Dam was approximately one-quarter of the total storage capacity of the Stony River Reservoir. Possibly the reduction in leakage in the former case has likewise been due to sedimentation; such an effect has frequently been observed by the writer.

The percentage of "plums" specified in connection with the Class *C* concrete was intended to fix a maximum limit, and does not represent an average of the quantity actually used. As a matter of fact, a maximum limit need hardly be fixed. At any rate, the writer agrees with Mr. Jorgensen that 20% of "plums" represents a very high average (page 1071). The writer does not know the actual quantity of "plums" placed in the Class *C* concrete at Stony River.

The mathematical solution presented (page 1076) by Professor Church, for determining portions of curves showing "discharge over spillways", such as used in Figs. 12 to 15, is indeed most interesting, utilizing, as it does, methods of attack in which Professor Church is a past master.

Unfortunately, quantitive measurements of leakage were not made on May 25th and November 1st, 1915, as referred to by Mr. O'Shaughnessy.

Mr. Justin offers an alternative suggestion for combining means to increase resistance to sliding and to increase the length of the path which leakage water from the reservoir must follow. The essentials of these means are shown in Fig. 39. With reference to the first phase of the suggestion, it may be said that preliminary studies along the same line, namely, of increasing the weight of superstructure (page 959), showed that nothing short of a complete filling of the available space with masonry or concrete would suffice. Of course, the considerations regarding overloading of the foundation soil at the toe were applied to the base as actually, and materially, widened by the building of the new toe-wall. Had the expedient of increasing the superstructure weight been adopted, it would naturally have been

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advisable, as Mr. Justin suggests, to place the added weight as far up stream as possible. Unfortunately, however, the slope of the deck would have precluded obtaining any material advantage thereby.

A similar method of transferring the load from the buttress into the toe-wall was actually used in the reconstruction in connection with the special work at the outlet gate bays (see Plate XIII).

As to the lengthening of the path of travel of leakage water, the writer believes that reliance may be placed on this method more safely in the case of gravel or sand foundations than where loam and clay are involved, as at the Stony River site. It should also be noted that the corresponding widening of the base of the dam, assuming an entire absence of drainage openings through the base, would increase the allowance necessary to be made for uplift pressure.

In the case of the suggested up-stream concrete apron, absolute water-tightness would be imperative, and therein, the writer believes, would lie the greatest difficulty, especially if the apron were made only 6 in. thick. Furthermore, the original cut-off wall is much less pervious than the soil which would underlie the suggested apron. Consequently, the result would probably be a gradual accumulation of water under pressure under the apron, with resulting decrease in the efficacy of the expedient. This would be true even if the reservoir water could penetrate under the apron only at its up-stream edge.

Although it was considered well worth while to make the expensive addition of anchoring walls to the original structure, it should be remembered that the same results could have been obtained much more easily and economically had a combination of anchoring wall at the heel and of cut-off been constructed as a part of the dam at the very beginning. For a case of original construction, therefore, the advantages of using the principle of anchoring walls are even more apparent.

The West Virginia Pulp and Paper Company is following the praiseworthy course of having annual inspections of the dam made by an experienced engineer; moreover, there is the additional safeguard of daily inspections by a regular attendant.

In conclusion, the writer wishes to express his satisfaction with the magnanimous approval contained in the discussions by Mr. Coburn, Chief Engineer of the Ambursen Company, and by Mr. Wegmann, who had been retained in an advisory capacity during the period when the original construction of the dam was under consideration.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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Paper No. 1398

FINAL REPORT OF THE SPECIAL COMMITTEE ON CONCRETE AND REINFORCED CONCRETE*

WITH DISCUSSION BY MESSRS. A. H. RHETT, WILLIAM FRY SCOTT, CARL GAYLER, H. V. HINCKLEY, HENRY T. EDDY, C. A. P. TURNER, F. E. TURNEAURE, AND A. N. TALBOT.†

TO THE PRESIDENT AND MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS:

GENTLEMEN.—Your Special Committee on Concrete and Reinforced Concrete herewith presents, as its Final Report, the result of the joint efforts of its own members and of the representatives of other Societies appointed for the same purpose.

Respectfully submitted,

JOSEPH R. WORCESTER,
Chairman,
RICHARD L. HUMPHREY,
Secretary,
J. E. GREINER,
W. K. HATT,
OLAF HOFF,
ROBERT W. LESLEY,
EMIL SWENSSON,
ARTHUR N. TALBOT.

NOVEMBER 17TH, 1916.

* Presented to the Annual Meeting, January 17th, 1917.

† Discussions on this report, by Messrs. L. J. Mensch, G. S. Bergendahl, and E. S. Martin were printed in the October, 1917, *Proceedings*, but are not published here because the Committee has not had sufficient time to consider and reply to them. These discussions, together with any reply the Committee may make, if so ordered by the Publication Committee, may be published later. It seems desirable, however, to publish the report in this volume, otherwise its publication would be delayed for a year.

FINAL REPORT

OF THE

JOINT COMMITTEE ON CONCRETE

AND REINFORCED CONCRETE

Preliminary Draft Prepared and Submitted by the Secretary, October 27th, 1908.

Amended and Adopted by Letter-Ballot of the Committee, December 20th, 1908.

Revised and Brought Up to Date, November 20th, 1912.

Final Report Adopted by the Committee, July 1st, 1916.

AFFILIATED COMMITTEES

OF THE

American Society of Civil Engineers,
American Society for Testing Materials,
American Railway Engineering Association,
Portland Cement Association,
American Concrete Institute.

July 1st, 1916.

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CHAPTER I.

Introduction.

The Joint Committee on Concrete and Reinforced Concrete was formed by the union of Special Committees appointed in 1903 and 1904 by the American Society of Civil Engineers, the American Society for Testing Materials, the American Railway Engineering and Maintenance of Way Association (now the American Railway Engineering Association), and the Association of American Portland Cement Manufacturers (now the Portland Cement Association). In 1915 there was added a Special Committee appointed by the American Concrete Institute at the invitation of the Joint Committee.

The present organization and membership of the Joint Committee is as follows:

Officers.

Chairman—Joseph R. Worcester.
Vice-Chairman—Emil Swensson.
Secretary—Richard L. Humphrey.

Members.

American Society of Civil Engineers.

John E. Greiner, Consulting Engineer, Baltimore, Md.
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PORTLAND CEMENT ASSOCIATION.

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AMERICAN CONCRETE INSTITUTE.

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EGBERT J. MOORE, Chief Engineer, Turner Construction Company, 11 Broadway, New York, N. Y.

LEONARD C. WASON, President, Aberthaw Construction Company, 27 School Street, Boston, Mass.

Progress reports by the Joint Committee were presented to the parent societies in 1909 and 1912. The report presented in 1912 has been printed by the American Society of Civil Engineers, the American Society for Testing Materials, and the American Railway Engineering Association, and reference to that report may be made for details regarding the earlier work of the Joint Committee, a historical sketch of the introduction of concrete and reinforced concrete, and a bibliography of authorities upon which the report was based.

The Committee has been much gratified at the reception accorded its 1912 report, and realizes the responsibility which rests upon it because of the very extensive adoption of its recommendations in current practice in this country. The members of the Committee are well aware of the incompleteness of that report, and even now they are unable to pass judgment upon some matters not dealt with in the present report.

Since 1912 the Committee has continued its study of the subject, has followed the working out of its recommendations in actual construction, has weighed arguments and criticisms which have come to its attention, and has considered new experimental data. While the Committee sees no reason for making any fundamental changes, the recommendations of its previous report have been revised to some extent, and considerable new material has been added upon subjects not previously touched. There are some subjects upon which experimentation is still in progress, and the art of concrete and reinforced concrete will be advancing for many years to come.

While this report deals with every kind of stress to which concrete is subjected, and includes all ordinary conditions of proportioning and handling, it does not go into all types of construction nor all the applications to which concrete and reinforced concrete may be put. The report is not a specification, but may be used as a basis for specifications. In their use, concrete and reinforced concrete involve the exercise of good judgment to a greater degree than do any other building materials. Rules cannot produce or supersede judgment; on the contrary, judgment should control the interpretation and application of rules.

The Committee has not attempted in every case to present rigidly scientific methods of analysis in dealing with stresses, but has aimed to furnish rules which will lead to safe results sufficiently close for ordinary design.

The Committee presumes that the application of the recommendations in this report to the design of any structure will be made only by persons having an adequate knowledge of the principles of structural design. Only persons with such knowledge and experience should be called upon to design reinforced concrete structures.

The Joint Committee has reached the conclusion that, with this effort to express the present state of the art, it would be desirable for it to withdraw from the field. This action has been taken in the hope that a work similar to that which the Committee has attempted to perform will again be undertaken, within a reasonable term of years, in order that there may be some authoritative body to consider and pass upon newly acquired knowledge and information gleaned from experience. The Committee feels certain, however, that it would be for the better interest of the profession to entrust this work to other hands rather than to continue the present organization with this object in view.

CHAPTER II.

Adaptability of Concrete and Reinforced Concrete.

The adaptability of concrete and reinforced concrete for engineering structures or parts thereof, is so well established that they are recognized materials of construction. When properly used, they have proved satisfactory for those purposes for which their qualities make them particularly suitable.

1. uses.

Plain concrete is well adapted for structures in which the principal stresses are compressive, such as: foundations, dams, retaining and other walls, tunnels, piers, abutments, and, in many cases, arches.

By the use of metal reinforcement to resist the principal tensile stresses, concrete becomes available for general use in various structures and structural forms. This combination of concrete and metal is particularly advantageous in structural members subject to both compression and tension, and in columns where, although the main stresses are compressive, there is also cross-bending.

Metal reinforcement may also be used to advantage to distribute and minimize cracks due to shrinkage and temperature changes.

2. precautions.

Failures of reinforced concrete structures have been due usually to some one or more of the following causes:

Defective design, poor material, faulty execution, or premature removal of forms.

To prevent failures or otherwise unsatisfactory results, the following precautions should be taken:

The computations and assumptions on which the design is based should be in accordance with the established principles of mechanics. The unit stresses and details of the design should conform to ac-

cepted good practice. Materials used for the concrete as well as for the reinforcement should be carefully inspected and tested, special attention being given to the testing of the sand, as poor sand has proved a frequent cause of failure. The measuring and combining of the materials which go to make up the concrete, and the placing of the concrete in the forms, should be under the supervision of experienced men. The metal for reinforcement should be of a quality conforming to standard specifications. Care should be taken to obtain good bond between different fills of concrete, to prevent concrete from freezing before the cement has set, to have the materials thoroughly mixed, to avoid too wet or too dry a consistency, and to have the forms cleaned before concrete is placed.

The computations should include all details; even minor details may be of the utmost importance. The design should show clearly the size and position of the reinforcement, and should provide for proper connection between the component parts so that they cannot be displaced. As the connections between reinforced concrete members are frequently a source of weakness, the design should include a detailed study of such connections.

The concrete should be rigidly supported until it has developed sufficient strength to carry imposed loads. The most careful and experienced inspection is necessary to determine when the concrete has set sufficiently for it to be safe to remove forms. Frozen concrete frequently has been mistaken for properly set concrete.

3. DESIGN AND SUPERVISION.

The execution of the work should not be separated from the design, as intelligent supervision and successful execution can be expected only when both functions are combined. It is desirable, therefore, that the engineer who prepares the design and specifications should have supervision of the execution of the work.

The Committee recommends the following practice for the purpose of fixing the responsibility and providing for adequate supervision during construction.

(*a*) Before work is commenced, complete plans and specifications should be prepared, giving the dead and live loads, wind and impact, if any, and working stresses, showing the general arrangement and all details. The plans should show the size, length, location of points of bending, and exact position of all reinforcement, including stirrups, ties, hooping, and splicing.

(*b*) The specifications should state the qualities of the materials and the proportions in which they are to be used.

(*c*) The strength which the concrete is expected to attain after a definite period should be stated in the specifications.

(*d*) Inspection during construction should be made by competent inspectors selected by and under the supervision of the engineer, and should cover the following:

1. Materials.
2. Construction and erection of the forms and supports.
3. Sizes, shapes, arrangement, position, and fastening of the reinforcement.
4. Proportioning, mixing, consistency, and placing of the concrete.
5. Strength of the concrete by tests of standard test pieces made on the work.
6. Whether the concrete is sufficiently hardened before the forms and supports are removed.
7. Protection from injury of all parts of the structure.
8. Comparison of dimensions of all parts of the finished structure with the plans.

(*e*) Load tests on portions of the finished structure should be made where there is reasonable suspicion that the work has not been properly performed, or that, through influences of some kind, the strength has been impaired, or where there is any doubt as to the sufficiency of the design. The loading should be carried to such a point that the calculated stresses under such loading shall be one and three-quarters times the allowed working stresses, and such loads should cause no injurious permanent deformations. Load tests should not be made before the concrete has been in place 60 days.

4. DESTRUCTIVE AGENCIES.

(*a*) *Corrosion of Metal Reinforcement.*—Tests and experience indicate that steel sufficiently embedded in good concrete is well protected against corrosion, no matter whether located above or below water level. It is recommended that such protection be not less than 1 in. in thickness. If the concrete is porous so as to be readily permeable by water, as when the concrete is laid with a very dry consistency, the metal may corrode on account of the presence of moisture and air.

(*b*) *Electrolysis.*—The experimental data available on this subject seem to show that while reinforced concrete structures may, under certain conditions, be injured by the flow of electric current in either direction between the reinforcing material and the concrete, such injury is generally to be expected only where voltages are considerably higher than those which usually occur in concrete structures in practice. If the iron be positive, trouble may manifest itself by corrosion of the iron accompanied by cracking of the concrete, and, if the iron be negative, there may be a softening of the concrete near the surface of the iron, resulting in a destruction of the bond. The former, or anode effect, decreases much more rapidly than the voltage, and almost

if not quite disappears at voltages that are most likely to be encountered in practice. The cathode effect, on the other hand, takes place even for very low voltages, and is therefore more important from a practical standpoint than that of the anode.

Structures containing salt or calcium chloride, even in very small quantities, are very much more susceptible to the effects of electric currents than normal concrete, the anode effect progressing much more rapidly in the presence of chlorine, and the cathode effect being greatly increased by the presence of an alkali metal.

There is great weight of evidence to show that normal reinforced concrete structures free from salt are in very little danger under most practical conditions, while non-reinforced concrete structures are practically immune from electrolysis troubles.

(*c*) *Sea Water.*—The data available concerning the effect of sea water on concrete or reinforced concrete are limited and inconclusive. Sea walls out of the range of frost action have been standing for many years without apparent injury. In many places serious disintegration has taken place. This has occurred chiefly between low and high tide levels, and is due, evidently, in part to frost. Chemical action also appears to be indicated by the softening of the mortar. To effect the best resistance to sea water, the concrete must be proportioned, mixed, and placed so as to prevent the penetration of sea water into the mass or through the joints. The aggregates should be carefully selected, graded, and proportioned with the cement so as to secure the maximum possible density; the concrete should be thoroughly mixed; the joints between old and new work should be made water-tight; and the concrete should be kept from exposure to sea water until it is thoroughly hard and impervious.

(*d*) *Acids.*—Dense concrete thoroughly hardened is affected appreciably only by acids which seriously injure other materials. Substances like manure, that contain acids, may injuriously affect green concrete, but do not affect concrete that is thoroughly hardened.

(*e*) *Oils.*—Concrete is unaffected by such mineral oils as petroleum and ordinary engine oils. Oils which contain fatty acids produce injurious effects, forming compounds with the lime which may result in a disintegration of the concrete in contact with them.

(*f*) *Alkalies.*—The action of alkalies on concrete is problematical. In the reclamation of arid land, where the soil is heavily charged with alkaline salts, it has been found that concrete, stone, brick, iron, and other materials are injured under certain conditions. It would seem that at the level of the ground-water in an extremely dry atmosphere such structures are disintegrated, through the rapid crystallization of the alkaline salts, resulting from the alternate wetting and drying of the surface. Such destructive action can be prevented by the use of a protective coating, and is minimized by securing a dense concrete.

CHAPTER III.

Materials.

The quality of all the materials is of paramount importance. The cement and also the aggregates should be subject to definite requirements and tests.

1. cement.

There are available for construction purposes Portland, Natural, and Puzzolan or Slag cements.

(*a*) *Portland Cement* is the product obtained by finely pulverizing clinker produced by calcining to incipient fusion, an intimate and properly proportioned mixture of argillaceous and calcareous materials, with no additions subsequent to calcination excepting water and calcined or uncalcined gypsum.

It has a definite chemical composition varying within comparatively narrow limits.

Portland cement only should be used in reinforced concrete construction or in any construction that will be subject to shocks, vibrations, or stresses other than direct compression.

(*b*) *Natural Cement* is the finely pulverized product resulting from the calcination of an argillaceous limestone at a temperature only sufficient to drive off the carbonic acid gas.

Although the limestone must have a certain composition, this composition may vary within much wider limits than in the case of Portland cement. Natural cement does not develop its strength as quickly nor is it as uniform in composition as Portland cement.

Natural cement may be used in massive masonry where weight rather than strength is the essential feature.

Where economy is the governing factor, a comparison may be made between the use of natural cement and a leaner mixture of Portland cement that will develop the same strength.

(*c*) *Puzzolan or Slag Cement* is the product resulting from finely pulverizing a mechanical mixture of granulated basic blast-furnace slag and hydrated lime.

Puzzolan cement is not nearly as strong, uniform, or reliable as Portland or natural cement, is not used extensively, and never in important work; it should be used only for unimportant foundation work underground where it is not exposed to air or running water.

(*d*) *Specifications.*—The cement should meet the requirements of the specifications and methods of tests for Portland cement, which are the result of the joint labors of Special Committees of the American Society of Civil Engineers, American Society for Testing Materials, American Railway Engineering Association, and other affiliated organizations, and the United States Government.

2. AGGREGATES.

Extreme care should be exercised in selecting the aggregates for mortar and concrete, and careful tests made of the materials for the purpose of determining the quality and grading necessary to secure maximum density[1] or a minimum percentage of voids. Bank gravel should be separated by screening into fine and coarse aggregates and then used in the proportions to be determined by density tests.

(*a*) *Fine Aggregate* should consist of sand, or the screenings of gravel or crushed stone, graded from fine to coarse, and passing when dry a screen having ¼-in. diameter holes;[2] it preferably should be of siliceous material, and not more than 30% by weight, should pass a sieve having 50 meshes per lin. in.; it should be clean, and free from soft particles, lumps of clay, vegetable loam or other organic matter.

Fine aggregate should always be tested for strength. It should be of such quality that mortar composed of one part Portland cement and three parts fine aggregate by weight when made into briquettes, prisms or cylinders will show a tensile or compressive strength, at an age of not less than 7 days, at least equal to the strength of 1:3 mortar of the same consistency made with the same cement and standard Ottawa sand.[3] If the aggregate be of poorer quality, the proportion of cement should be increased to secure the desired strength. If the strength developed by the aggregate in the 1:3 mortar is less than 70% of the strength of the Ottawa-sand mortar, the material should be rejected. In testing aggregates care should be exercised to avoid the removal of any coating on the grains, which may affect the strength; bank sands should not be dried before being made into mortar, but should contain natural moisture. The percentage of moisture may be determined upon a separate sample for correcting weight. From 10 to 40% more water may be required in mixing bank or artificial sands than for standard Ottawa sand to produce the same consistency.

(*b*) *Coarse Aggregate* should consist of gravel or crushed stone which is retained on a screen having ¼-in. diameter holes, and should be graded from the smallest to the largest particles; it should be clean, hard, durable, and free from all deleterious matter. Aggregates containing dust and soft, flat, or elongated particles, should be excluded. The Committee does not feel warranted in recommending the use of blast-furnace slag as an aggregate, in the absence of adequate data as to its value, especially in reinforced concrete construction. No

[1] A convenient coefficient of density is the ratio of the volumes of solid particles contained in a unit volume to the total unit volume.

[2] If the dividing size between the fine and coarse aggregate is less or greater than ¼ in., allowance should be made in grading and proportioning.

[3] A natural sand obtained at Ottawa, Illinois, passing a screen having 20 meshes and retained on a screen having 30 meshes per linear inch; prepared and furnished by the Ottawa Silica Company, for 2 cents per pound f. o. b. cars, Ottawa, Illinois.

satisfactory specifications or methods of inspection have been developed that will control its uniformity and ensure the durability of the concrete in which it is used.

The aggregate must be small enough to produce with the mortar a homogeneous concrete of sluggish consistency which will pass readily between and easily surround the reinforcement and fill all parts of the forms. The maximum size of particles is variously determined for different types of construction from that which will pass a ¼-in. ring to that which will pass a 1½-in. ring.

For concrete in large masses the size of the coarse aggregate may be increased, as a large aggregate produces a stronger concrete than a fine one; however, it should be noted that the danger of separation from the mortar becomes greater as the size of the coarse aggregate increases.

Cinder concrete should not be used for reinforced concrete structures, except in floor-slabs not exceeding 8-ft. span. It also may be used for fire protection purposes where not required to carry loads. The cinders used should be composed of hard, clean, vitreous clinker, free from sulphides, unburned coal, or ashes.

3. WATER.

The water used in mixing concrete should be free from oil, acid, alkali, or organic matter.

4. METAL REINFORCEMENT.

The Committee recommends as a suitable material for reinforcement, steel of structural grade filling the requirements of the Specifications for Billet Steel Concrete Reinforcement Bars of the American Society for Testing Materials.

For reinforcing slabs, small beams, or minor details, or for reinforcing for shrinkage and temperature stresses, steel wire, expanded metal, or other reticulated steel may be used, with the unit stresses hereinafter recommended.

The reinforcement should be free from flaking, rust, scale, or coatings of any character which would tend to reduce or destroy the bond.

CHAPTER IV.

Mixing and Placing.

1. PROPORTIONS.

The materials should be carefully selected, of uniform quality, and proportioned with a view to securing as nearly as possible a maximum density, which is obtained by grading the aggregates so that the smaller particles fill the spaces between the larger, thus reducing the voids in the aggregate to the minimum.

(*a*) *Unit of Measure.*—The measurement of the fine and coarse aggregates should be by loose volume. The unit of measure should be a bag of cement, containing 94 lb. net, which should be considered the equivalent of 1 cu. ft.

(*b*) *Relation of Fine and Coarse Aggregates.*—The fine and coarse aggregates should be used in such proportions as will secure maximum density. These proportions should be carefully determined by density experiments, and the grading of the fine and coarse aggregates should be uniformly maintained, or the proportions changed, to meet the varying sizes.

(*c*) *Relation of Cement and Aggregates.*—For reinforced concrete construction, one part of cement to a total of six parts of fine and coarse aggregates measured separately should generally be used. For columns, richer mixtures are preferable. In massive masonry or rubble concrete, a mixture of 1:9 or even 1:12 may be used.

These proportions should be determined by the strength or other qualities required in the construction at the critical period of use. Experience and judgment based on observation and tests of similar conditions in similar localities are excellent guides as to the proper proportions for any particular case.

In important construction, advance tests should be made on concrete composed of the materials to be used in the work. These tests should be made by standardized methods, to obtain uniformity in mixing, proportioning, and storage, and in case the results do not conform to the requirements of the work, aggregates of a better quality or more cement should be used to obtain the desired quality of concrete.

2. MIXING.

The mixing of concrete should be thorough, and continue until the mass is uniform in color and homogeneous. As the maximum density and greatest strength of a given mixture depend largely on thorough and complete mixing, it is essential that this part of the work should receive special attention and care.

Inasmuch as it is difficult to determine, by visual inspection, whether the concrete is uniformly mixed, especially where aggregates having the color of cement are used, it is essential that the mixing should occupy a definite period of time. The minimum time will depend on whether the mixing is done by machine or hand.

(*a*) *Measuring Ingredients.*—Methods of measurement of the various ingredients should be used which will secure at all times separate and uniform measurements of cement, fine aggregate, coarse aggregate, and water.

(*b*) *Machine Mixing.*—The mixing should be done in a batch machine mixer of a type which will ensure the uniform distribution of

the materials throughout the mass, and should continue for the minimum time of 1½ min. after all the ingredients are assembled in the mixer. For mixers of two or more cubic yards capacity, the minimum time of mixing should be 2 min. Since the strength of the concrete is dependent upon thorough mixing, a longer time than this minimum is preferable. It is desirable to have the mixer equipped with an attachment for automatically locking the discharging device so as to prevent the emptying of the mixer until all the materials have been mixed together for the minimum time required after they are assembled in the mixer. Means should be provided to prevent aggregates being added after the mixing has commenced. The mixer should also be equipped with water storage, and an automatic measuring device which can be locked is desirable. It is also desirable to equip the mixer with a device recording the revolutions of the drum. The number of revolutions should be so regulated as to give at the periphery of the drum a uniform speed; about 200 ft. per min. seems to be the best speed in the present state of the art.

(*c*) *Hand Mixing.*—Hand mixing should be done on a water-tight platform and especial precautions taken after the water has been added, to turn all the ingredients together at least six times, and until the mass is homogeneous in appearance and color.

(*d*) *Consistency.*—The materials should be mixed wet enough to produce a concrete of such a consistency as will flow sluggishly into the forms and about the metal reinforcement when used, and which, at the same time, can be conveyed from the mixer to the forms without separation of the coarse aggregate from the mortar. The quantity of water is of the greatest importance in securing concrete of maximum strength and density; too much water is as objectionable as too little.

(*e*) *Retempering.*—The remixing of mortar or concrete that has partly set should not be permitted.

3. PLACING CONCRETE.

(*a*) *Methods.*—Concrete after the completion of the mixing should be conveyed rapidly to the place of final deposit; under no circumstances should concrete be used that has partly set.

Concrete should be deposited in such a manner as will permit the most thorough compacting, such as can be obtained by working with a straight shovel or slicing tool kept moving up and down until all the ingredients are in their proper place. Special care should be exercised to prevent the formation of laitance; where laitance has formed it should be removed, since it lacks strength, and prevents a proper bond in the concrete.

Before depositing concrete, the reinforcement should be carefully placed in accordance with the plans. It is essential that adequate

means be provided to hold it in its proper position until the concrete has been deposited and compacted; care should be taken that the forms are substantial and thoroughly wetted (except in freezing weather) or oiled, and that the space to be occupied by the concrete is free from débris. When the placing of concrete is suspended, all necessary grooves for joining future work should be made before the concrete has set.

When work is resumed, concrete previously placed should be roughened, cleansed of foreign material and laitance, thoroughly wetted and then slushed with a mortar consisting of one part Portland cement and not more than two parts fine aggregate.

The surfaces of concrete exposed to premature drying should be kept covered and wet for a period of at least 7 days.

Where concrete is conveyed by spouting, the plant should be of such a size and design as to ensure a practically continuous stream in the spout. The angle of the spout with the horizontal should be such as to allow the concrete to flow without a separation of the ingredients; in general an angle of about 27° or one vertical to two horizontal is good practice. The spout should be thoroughly flushed with water before and after each run. The delivery from the spout should be as close as possible to the point of deposit. Where the discharge must be intermittent, a hopper should be provided at the bottom. Spouting through a vertical pipe is satisfactory when the flow is continuous; when it is unchecked and discontinuous it is highly objectionable, unless the flow is checked by baffle plates.

(*b*) *Freezing Weather.*—Concrete should not be mixed or deposited at a freezing temperature, unless special precautions are taken to prevent the use of materials covered with ice crystals or containing frost, and to prevent the concrete from freezing before it has set and sufficiently hardened.

As the coarse aggregate forms the greater portion of the concrete, it is particularly important that this material be warmed to well above the freezing point.

The enclosing of a structure and the warming of the space inside the enclosure is recommended, but the use of salt to lower the freezing point is not recommended.

(*c*) *Rubble Concrete.*—Where the concrete is to be deposited in massive work, its value may be improved and its cost materially reduced by the use of clean stones, saturated with water, thoroughly embedded in and entirely surrounded by concrete.

(*d*) *Under Water.*—In placing concrete under water, it is essential to maintain still water at the place of deposit. With careful inspection, the use of tremies, properly designed and operated, is a satisfactory method of placing concrete through water. The concrete should be mixed very wet (more so than is ordinarily permissible) so that it

will flow readily through the tremie and into place with practically a level surface.

The coarse aggregate should be smaller than ordinarily used, and never more than 1 in. in diameter. The use of gravel facilitates mixing and assists the flow. The mouth of the tremie should be buried in the concrete so that it is at all times entirely sealed and the surrounding water prevented from forcing itself into the tremie; the concrete will then discharge without coming in contact with the water. The tremie should be suspended so that it can be lowered quickly when it is necessary either to choke off or prevent too rapid flow; the lateral flow preferably should be not over 15 ft.

The flow should be continuous in order to produce a monolithic mass and to prevent the formation of laitance in the interior.

In case the flow is interrupted, it is important that all laitance be removed before proceeding with the work.

In large structures it may be necessary to divide the mass of concrete into several small compartments or units, to permit the continuous filling of each one. With proper care it is possible in this manner to obtain as good results under water as in the air.

A less desirable method is the use of the drop-bottom bucket. Where this method is used, the bottom of the bucket should be released when in contact with the surface of the place of deposit.

CHAPTER V.

Forms.

Forms should be substantial and unyielding, in order that the concrete may conform to the design, and be sufficiently tight to prevent the leakage of mortar.

It is vitally important to allow sufficient time for the proper hardening of the concrete, which should be determined by careful inspection before the forms are removed.

Many conditions affect the hardening of concrete, and the proper time for the removal of the forms should be determined by some competent and responsible person.

It may be stated in a general way that forms should remain in place longer for reinforced concrete than is required for plain or massive concrete, and longer for horizontal than is required for vertical members.

In general it may be considered that concrete has hardened sufficiently when it has a distinctive ring under the blow of a hammer, but this test is not reliable, if there is a possibility that the concrete is frozen.

CHAPTER VI.

Details of Construction.

1. Joints.

(*a*) *In Concrete.*—It is desirable to cast an entire structure at one operation, but as this is not always possible, especially in large structures, it is necessary to stop the work at some convenient point. This should be selected so that the resulting joint may have the least possible effect on the strength of the structure. It is therefore recommended that the joint in columns be made flush with the lower side of the girders, or in flat slab construction at the bottom of the flare of the column head; that the joints in girders be at a point midway between supports, unless a beam intersects a girder at this point, in which case the joint should be offset a distance equal to twice the width of the beam; and that the joints in the members of a floor system should in general be made at or near the center of the span.

Joints in columns should be perpendicular to the axis, and in girders, beams, and floor-slabs, perpendicular to the plane of their surfaces. When it is necessary to provide for shear at right angles to the axis, it is permissible to incline the plane of the joint as much as 30° from the perpendicular. Joints in arch rings should be on planes as nearly radial as practicable.

Before placing the concrete on top of a freshly poured column a period of at least 2 hours should be allowed for the settlement and shrinkage.

Shrinkage and contraction joints may be necessary to concentrate cracks due to temperature in smooth even lines. The number of these joints which should be determined and provided for in the design will depend on the range of temperature to which the concrete will be subjected, and on the amount and position of the reinforcement. In massive work, such as retaining walls, abutments, etc., built without reinforcement, contraction joints should be provided, at intervals of from 25 to 50 ft. and with reinforcement from 50 to 80 ft.; the smaller the height and thickness, the closer the spacing. The joints should be tongued and grooved to maintain the alignment in case of unequal settlement. A groove may be formed in the surface as a finish to vertical joints.

Shrinkage and contraction joints should be lubricated by an application of petroleum oil or a similar material, to permit a free movement when the concrete expands or contracts.

The movement of the joint due to expansion and contraction may be facilitated by the insertion of a sheet of copper, zinc, or even tarred paper.

(*b*) *In Reinforcement.*—Wherever it is necessary to splice tension reinforcement the length of lap should be determined on the basis of

the safe bond stress, the stress in the bar and the shearing resistance of the concrete at the point of splice; or a connection should be made between the bars of sufficient strength to carry the stress. Splices at points of maximum stress in tension should be avoided. In columns, bars more than $\frac{3}{4}$ in. in diameter not subject to tension should have their ends properly squared and butted together in suitable sleeves; smaller bars may be lapped, as indicated for tension reinforcement. At foundations bearing plates should be provided for supporting the bars, or the bars may be carried into the footing a sufficient distance to transmit the stress in the steel to the concrete by means of the bearing and the bond resistance. In no case should reliance be placed upon the end bearing of bars on concrete.

2. SHRINKAGE AND TEMPERATURE CHANGES.

The stresses resulting from shrinkage due to hardening and contraction from temperature changes are important in monolithic construction, and unless cared for in the design will produce objectionable cracks; cracks cannot be entirely prevented, but the effects can be minimized.

Large cracks, produced by quick hardening or wide ranges of temperature, can be broken up to some extent into small cracks by placing reinforcement in the concrete; in long, continuous lengths of concrete, it is better to provide shrinkage joints at points in the structure where they will do little or no harm. Reinforcement is of assistance, and permits longer distances between shrinkage joints than when no reinforcement is used.

Provision for shrinkage should be made when small or thin masses are joined to larger or thicker masses; at such places the use of fillets similar to those used in metal castings, but proportionally larger, is recommended.

Shrinkage cracks are likely to occur at points where fresh concrete is joined to that which is set, and hence in placing the concrete, construction joints should be made, as described in Chapter VI, Section 1, or, if possible, at points where joints would naturally occur in dimension-stone masonry.

3. FIRE-PROOFING.

Concrete, because incombustible and of a low rate of heat conductivity, is highly efficient and admirably adapted for fire-proofing purposes. This has been demonstrated by experience and tests.

The dehydration of concrete probably begins at about 500° Fahr. and is completed at about 900° Fahr., but experience indicates that the volatilization of the water absorbs heat from the surrounding mass, which, together with the resistance of the air cells, tends to increase the heat resistance of the concrete, so that the process of dehy-

dration is very much retarded. The concrete that is actually affected by fire and remains in position affords protection to that beneath it.

The thickness of the protective coating should be governed by the intensity and duration of a possible fire and the rate of heat conductivity of the concrete. The question of the rate of heat conductivity of concrete is one which requires further study and investigation before a definite rate for different classes of concrete can be fully established. However, for ordinary conditions, it is recommended that the metal be protected by a minimum of 2 in. of concrete on girders and columns, 1½ in. on beams, and 1 in. on floor-slabs.

Where fire-proofing is required, and not otherwise provided in monolithic concrete columns, it is recommended that the concrete to a depth of 1½ in. be considered as protective covering, and not included in the effective section.

The corners of columns, girders, and beams should be beveled or rounded, as a sharp corner is more seriously affected by fire than a round one; experience shows that round columns are more fire resistive than square.

4. WATER-PROOFING.

Many expedients have been resorted to for rendering concrete impervious to water. Experience shows, however, that when mortar or concrete is proportioned to obtain the greatest practicable density and is mixed to the proper consistency (Chapter IV, Section 2 *d*), the resulting mortar or concrete is impervious under moderate pressure.

On the other hand, concrete of dry consistency is more or less pervious to water, and, though compounds of various kinds have been mixed with the concrete or applied as a wash to the surface, in an effort to offset this defect, these expedients have generally been disappointing, for the reason that many of these compounds have at best but temporary value, and in time lose their power of imparting impermeability to the concrete.

In the case of subways, long retaining walls and reservoirs, provided the concrete itself is impervious, cracks may be so reduced, by horizontal and vertical reinforcement properly proportioned and located, that they will be too minute to permit leakage, or will be closed by infiltration of silt.

Asphaltic or coal-tar preparations applied either as a mastic or as a coating on felt or cloth fabric, are used for water-proofing, and should be proof against injury by liquids or gases.

For retaining and similar walls in direct contact with the earth, the application of one or two coatings of hot coal-tar pitch, following a painting with a thin wash of coal tar dissolved in benzol, to the thoroughly dried surface of concrete is an efficient method of preventing the penetration of moisture from the earth.

5. SURFACE FINISH.

Concrete is a material of an individual type, and should be used without effort at imitation of other building materials. One of the important problems connected with its use is the character of the finish of exposed surfaces. The desired finish should be determined before the concrete is placed, and the work conducted so as to facilitate securing it. The natural surface of the concrete in most structures is unobjectionable, but in others the marks of the forms and the flat dead surface are displeasing, making some special treatment desirable. A treatment of the surface which removes the film of cement and brings the aggregates of the concrete into relief, either by scrubbing with brushes and water before it is hard, or by tooling it after it is hard, is frequently used to erase the form markings and break the monotonous appearance of the surface. Besides being more pleasing in immediate appearance, such a surface is less subject to discoloration and hair cracking than is a surface composed of the cement that segregates against the forms, or one that is made by applying a cement wash. The aggregates can also be exposed by washing with hydrochloric acid diluted with from 6 to 10 parts of water. The plastering of surfaces should be avoided, for even if carefully done, it is liable to peel off under the action of frost or temperature changes.

Various effects in texture and in color can be obtained when the surface is to be scrubbed or tooled, by using aggregates of the desired size and color. For a fine-grained texture a granolithic surface mixture can be made and placed against the face forms to a thickness of about 1 in. as the placing of the body of the concrete proceeds.

A smooth, even surface without form marks can be secured by the use of plastered forms, which, in structures having many duplications of members, can be used repeatedly; these are made in panels of expanded metal or wire mesh coated with plaster, and the joints made at edges, and closed with plaster of Paris.

CHAPTER VII.

DESIGN.

1. MASSIVE CONCRETE.

In the design of massive or plain concrete, no account should be taken of the tensile strength of the material, and sections should usually be proportioned so as to avoid tensile stresses except in slight amounts to resist indirect stresses. This will generally be accomplished in the case of rectangular shapes if the line of pressure is kept within the middle third of the section, but in very large structures, such as high masonry dams, a more exact analysis may be required. Structures of massive concrete are able to resist unbalanced lateral

forces by reason of their weight; hence the element of weight rather than strength often determines the design. A leaner and relatively cheap concrete, therefore, will often be suitable for massive concrete structures.

It is desirable generally to provide joints at intervals to localize the effect of contraction (Chapter VI, Section 1).

Massive concrete is suitable for dams, retaining walls, and piers in which the ratio of length to least width is relatively small. Under ordinary conditions, this ratio should not exceed four. It is also suitable for arches of moderate span.

2. REINFORCED CONCRETE.

The use of metal reinforcement is particularly advantageous in members such as beams in which both tension and compression exist, and in columns where the principal stresses are compressive and where there also may be cross-bending. Therefore the theory of design here presented relates mainly to the analysis of beams and columns.

3. GENERAL ASSUMPTIONS.

(*a*) *Loads.*—The forces to be resisted are those due to:

1. *The dead load,* which includes the weight of the structure and fixed loads and forces.
2. *The live load,* or the loads and forces which are variable. The dynamic effect of the live load will often require consideration. Allowance for the latter is preferably made by a proportionate increase in either the live load or the live-load stresses. The working stresses hereinafter recommended are intended to apply to the equivalent static stresses thus determined.

   In the case of high buildings, the live load on columns may be reduced in accordance with the usual practice.

(*b*) *Lengths of Beams and Columns.*—The span length for beams and slabs simply supported should be taken as the distance from center to center of supports, but need not be taken to exceed the clear span plus the depth of beam or slab. For continuous or restrained beams built monolithically into supports, the span length may be taken as the clear distance between faces of supports. Brackets should not be considered as reducing the clear span in the sense here intended, except that when brackets which make an angle of 45° or more with the axis of a restrained beam are built monolithically with the beam, the span may be measured from the section where the combined depth of beam and bracket is at least one-third more than the depth of the beam. Maximum negative moments are to be considered as existing at the end of the span as here defined.

When the depth of a restrained beam is greater at its ends than at mid-span and the slope of the bottom of the beam at its ends makes an angle of not more than 15° with the direction of the axis of the beam at mid-span, the span length may be measured from face to face of supports.

The length of columns should be taken as the maximum unstayed length.

(*c*) *Stresses.*—The following assumptions are recommended as a basis for calculations:

1. Calculations will be made with reference to working stresses and safe loads, rather than with reference to ultimate strength and ultimate loads.
2. A plane section before bending remains plane after bending.
3. The modulus of elasticity of concrete in compression is constant within the usual limits of working stresses. The distribution of compressive stress in beams is therefore rectilinear.
4. In calculating the moment of resistance of beams, the tensile stresses in the concrete are neglected.
5. The adhesion between the concrete and the reinforcement is perfect. Under compressive stress the two materials are therefore stressed in proportion to their moduli of elasticity.
6. The ratio of the modulus of elasticity of steel to the modulus of elasticity of concrete is taken at 15 except as modified in Chapter VIII, Section 8.
7. Initial stress in the reinforcement due to contraction or expansion of the concrete is neglected.

It is recognized that some of the assumptions given herein are not entirely borne out by experimental data. They are given in the interest of simplicity and uniformity, and variations from exact conditions are taken into account in the selection of formulas and working stresses.

The deflection of a beam depends upon the strength and stiffness developed throughout its length. For calculating deflection, a value of 8 for the ratio of the moduli will give results corresponding approximately with the actual conditions.

4. T-BEAMS.

In beam and slab construction an effective bond should be provided at the junction of the beam and slab. When the principal slab reinforcement is parallel to the beam, transverse reinforcement should be used, extending over the beam and well into the slab.

The slab may be considered an integral part of the beam, when adequate bond and shearing resistance between slab and web of beam is provided, but its effective width shall be determined by the following rules:

(*a*) It shall not exceed one-fourth of the span length of the beam;

(*b*) Its overhanging width on either side of the web shall not exceed six times the thickness of the slab.

In the design of continuous T-beams, due consideration should be given to the compressive stress at the support.

Beams in which the T-form is used only for the purpose of providing additional compression area of concrete should preferably have a width of flange not more than three times the width of the stem and a thickness of flange not less than one-third of the depth of the beam. Both in this form and in the beam and slab form the web stresses and the limitations in placing and spacing the longitudinal reinforcement will probably be controlling factors in design.

5. FLOOR-SLABS SUPPORTED ALONG FOUR SIDES.

Floor-slabs having the supports extending along the four sides should be designed and reinforced as continuous over the supports. If the length of the slab exceeds one and one-half times its width, the entire load should be carried by transverse reinforcement.

For uniformly distributed loads on square slabs, one-half the live and dead load may be used in the calculations of moment to be resisted in each direction. For oblong slabs, the length of which is not greater than one and one-half times their width, the moment to be resisted by the transverse reinforcement may be found by using a proportion of the live and dead load equal to that given by the formula, $r = \frac{l}{b} - 0.5$, where l = length and b = breadth of slab. The longitudinal reinforcement should then be proportioned to carry the remainder of the load.

In placing reinforcement in such slabs account may well be taken of the fact that the bending moment is greater near the center of the slab than near the edges. For this purpose two-thirds of the previously calculated moments may be assumed as carried by the center half of the slab and one-third by the outside quarters.

Loads carried to beams by slabs which are reinforced in two directions will not be uniformly distributed to the supporting beams, and the distribution will depend on the relative stiffness of the slab and the supporting beams. The distribution which may be expected ordinarily is a variation of the load in the beam in accordance with

the ordinates of a parabola, having its vertex at the middle of the span. For any given design, the probable distribution should be ascertained and the moments in the beam calculated accordingly.

6. CONTINUOUS BEAMS AND SLABS.

When the beam or slab is continuous over its supports, reinforcement should be fully provided at points of negative moment, and the stresses in concrete recommended in Chapter VIII, Section 4, should not be exceeded. In computing the positive and negative moments in beams and slabs continuous over several supports, due to uniformly distributed loads, the following rules are recommended:

(*a*) For floor-slabs, the bending moments at center and at support should be taken at $\frac{wl^2}{12}$ for both dead and live loads, where w represents the load per linear unit and l the span length.

(*b*) For beams, the bending moment at center and at support for interior spans should be taken at $\frac{wl^2}{12}$ and for end spans it should be taken at $\frac{wl^2}{10}$ for center and interior support, for both dead and live loads.

(*c*) In the case of beams and slabs continuous for two spans only, with their ends restrained, the bending moment both at the central support and near the middle of the span should be taken as $\frac{wl^2}{10}$.

(*d*) At the ends of continuous beams, the amount of negative moment which will be developed in the beam will depend on the condition of restraint or fixedness, and this will depend on the form of construction used. In the ordinary cases a moment of $\frac{wl^2}{16}$ may be taken; for small beams running into heavy columns this should be increased, but not to exceed $\frac{wl^2}{12}$.

For spans of unusual length, or for spans of materially unequal length, more exact calculations should be made. Special consideration is also required in the case of concentrated loads.

Even if the center of the span is designed for a greater bending moment than is called for by (*a*) or (*b*), the negative moment at the support should not be taken as less than the values there given.

Where beams are reinforced on the compression side, the steel may be assumed to carry its proportion of stress in accordance with the ratio of moduli of elasticity, Chapter VIII, Section 8. Reinforcing bars for compression in beams should be straight and should be two diameters in the clear from the surface of the concrete. For the positive bending moment, such reinforcement should not exceed 1% of the area of the concrete. In the case of cantilever and continuous beams, tensile and compressive reinforcement over supports should extend sufficiently beyond the support and beyond the point of inflection to develop the requisite bond strength.

In construction made continuous over supports, it is important that ample foundations should be provided; for unequal settlements are liable to produce unsightly if not dangerous cracks. This effect is more likely to occur in low structures.

Girders, such as wall girders, which have beams framed into one side only, should be designed to resist torsional moment arising from the negative moment at the end of the beam.

7. BOND STRENGTH AND SPACING OF REINFORCEMENT.

Adequate bond strength should be provided. The formula hereinafter given for bond stresses in beams is for straight longitudinal bars. In beams in which a portion of the reinforcement is bent up near the end, the bond stress at places, in both the straight bars and the bent bars, will be considerably greater than for all the bars straight, and the stress at some point may be several times as much as that found by considering the stress to be uniformly distributed along the bar. In restrained and cantilever beams, full tensile stress exists in the reinforcing bars at the point of support, and the bars should be anchored in the support sufficiently to develop this stress.

In case of anchorage of bars, an additional length of bar should be provided beyond that found on the assumption of uniform bond stress, for the reason that before the bond resistance at the end of the bar can be developed the bar may have begun to slip at another point, and "running" resistance is less than the resistance before slip begins.

Where high bond resistance is required, the deformed bar is a suitable means of supplying the necessary strength. But it should be recognized that, even with a deformed bar, initial slip occurs at early loads, and that the ultimate loads obtained in the usual tests for bond resistance may be misleading. Adequate bond strength throughout the length of a bar is preferable to end anchorage, but, as an additional safeguard, such anchorage may properly be used in special cases. Anchorage furnished by short bends at a right angle is less effective than by hooks consisting of turns through 180 degrees.

The lateral spacing of parallel bars should be not less than three diameters from center to center, nor should the distance from the side of the beam to the center of the nearest bar be less than two diameters. The clear spacing between two layers of bars should be not less than 1 in. The use of more than two layers is not recommended, unless the layers are tied together by adequate metal connections, particularly at and near points where bars are bent up or bent down. Where more than one layer is used, at least all bars above the lower layer should be bent up and anchored beyond the edge of the support.

8. DIAGONAL TENSION AND SHEAR.

When a reinforced concrete beam is subjected to flexural action, diagonal tensile stresses are set up. A beam without web reinforcement will fail if these stresses exceed the tensile strength of the concrete. When web reinforcement, made up of stirrups or of diagonal bars secured to the longitudinal reinforcement, or of longitudinal reinforcing bars bent up at several points, is used, new conditions prevail, but, even in this case, at the beginning of loading the diagonal tension developed is taken principally by the concrete, the deformations which are developed in the concrete permitting but little stress to be taken by the web reinforcement. When the resistance of the concrete to the diagonal tension is overcome at any point in the depth of the beam, greater stress is at once set up in the web reinforcement.

For homogeneous beams, the analytical treatment of diagonal tension is not very complex—the diagonal tensile stress is a function of the horizontal and vertical shearing stresses and of the horizontal tensile stress at the point considered, and as the intensity of these three stresses varies from the neutral axis to the remotest fiber, the intensity of the diagonal tension will be different at different points in the section, and will change with different proportionate dimensions of length to depth of beam. For the composite structure of reinforced concrete beams, an analysis of the web stresses, and particularly of the diagonal tensile stresses, is very complex; and when the variations due to a change from no horizontal tensile stress in the concrete at remotest fiber to the presence of horizontal tensile stress at some point below the neutral axis are considered, the problem becomes more complex and indefinite. Under these circumstances, in designing, recourse is had to the use of the calculated vertical shearing stress, as a means of comparing or measuring the diagonal tensile stresses developed, it being understood that the vertical shearing stress is not the numerical equivalent of the diagonal tensile stress, and that there is not even a constant ratio between them. It is here recommended that the maximum vertical shearing stress in a section be used as the means of

comparison of the resistance to diagonal tensile stress developed in the concrete in beams not having web reinforcement.

Even after the concrete has reached its limit of resistance to diagonal tension, if the beam has web reinforcement, conditions of beam action will continue to prevail, at least through the compression area, and the web reinforcement will be called on to resist only a part of the web stresses. From experiments with beams it is concluded that it is safe practice to use only two-thirds of the external vertical shear in making calculations of the stresses that come on stirrups, diagonal web pieces, and bent-up bars, and it is here recommended for calculations in designing that two-thirds of the external vertical shear be taken as producing stresses in web reinforcement.

It is well established that vertical members attached to or looped about horizontal members, inclined members secured to horizontal members in such a way as to insure against slip, and the bending of a part of the longitudinal reinforcement at an angle, will increase the strength of a beam against failure by diagonal tension, and that a well-designed and well-distributed web reinforcement may, under the best conditions, increase the total vertical shear carried to a value as much as three times that obtained when the bars are all horizontal and no web reinforcement is used.

When web reinforcement comes into action as the principal tension web resistance, the bond stresses between the longitudinal bars and the concrete are not distributed as uniformly along the bars as they otherwise would be, but tend to be concentrated at and near stirrups, and at and near the points where bars are bent up. When stirrups are not rigidly attached to the longitudinal bars, and the proportioning of bars and stirrup spacing is such that local slip of bars occur at stirrups, the effectiveness of the stirrups is impaired, though the presence of stirrups still gives an element of toughness against diagonal tension failure.

Sufficient bond resistance between the concrete and the stirrups or diagonals must be provided in the compression area of the beam.

The longitudinal spacing of vertical stirrups should not exceed one-half the depth of beam, and that of inclined members should not exceed three-fourths of the depth of beam.

Bending of longitudinal reinforcing bars at an angle across the web of the beam may be considered as adding to diagonal tension resistance for a horizontal distance from the point of bending equal to three-fourths of the depth of beam. Where the bending is made at two or more points, the distance between points of bending should not exceed three-fourths of the depth of the beam. In the case of a restrained beam, the effect of bending up a bar at the bottom of the beam in resisting diagonal tension may not be taken as extending

beyond a section at the point of inflection, and the effect of bending down a bar in the region of negative moment may be taken as extending from the point of bending down of bar nearest the support to a section not more than three-fourths of the depth of beam beyond the point of bending down of bar farthest from the support, but not beyond the point of inflection. In case stirrups are used in the beam away from the region in which the bent bars are considered effective, a stirrup should be placed not farther than a distance equal to one-fourth the depth of beam from the limiting sections defined above. In case the web resistance required through the region of bent bars is greater than that furnished by the bent bars, sufficient additional web reinforcement in the form of stirrups or attached diagonals should be provided. The higher resistance to diagonal tension stresses given by unit frames having the stirrups and bent-up bars securely connected together both longitudinally and laterally is worthy of recognition. It is necessary that a limit be placed on the amount of shear which may be allowed in a beam; for when web reinforcement sufficiently efficient to give very high web resistance is used, at the higher stresses the concrete in the beam becomes checked and cracked in such a way as to endanger its durability as well as its strength.

The section to be taken as the critical section in the calculation of shearing stresses will generally be the one having the maximum vertical shear, though experiments show that the section at which diagonal tension failures occur is not just at a support, even though the shear at the latter point be much greater.

In the case of restrained beams, the first stirrup or the point of bending down of bar should be placed not farther than one-half of the depth of beam away from the face of the support.

It is important that adequate bond strength or anchorage be provided to develop fully the assumed strength of all web reinforcement.

Low bond stresses in the longitudinal bars are helpful in giving resistance against diagonal tension failures, and anchorage of longitudinal bars at the ends of the beams or in the supports is advantageous.

It should be noted that it is on the tension side of a beam that diagonal tension develops in a critical way, and that proper connection should always be made between stirrups or other web reinforcement and the longitudinal tension reinforcement, whether the latter is on the lower side of the beam or on its upper side. Where negative moment exists, as is the case near the supports in a continuous beam, web reinforcement, to be effective, must be looped over or wrapped around, or be connected with, the longitudinal tension reinforcing bars at the top of the beam in the same way as is necessary at the bottom of the beam at sections where the bending moment is positive.

Inasmuch as the smaller the longitudinal deformations in the

horizontal reinforcement are, the less the tendency for the formation of diagonal cracks, a beam will be strengthened against diagonal tension failure by so arranging and proportioning the horizontal reinforcement that the unit stresses at points of large shear shall be relatively low.

It does not seem feasible to make a complete analysis of the action of web reinforcement, and more or less empirical methods of calculation are therefore employed. Limiting values of working stresses for different types of web reinforcement are given in Chapter VIII, Section 5. The conditions apply to cases commonly met in design. It is assumed that adequate bond resistance or anchorage of all web reinforcement will be provided.

When a flat slab rests on a column, or a column bears on a footing, the vertical shearing stresses in the slab or footing immediately adjacent to the column are termed punching shearing stresses. The element of diagonal tension, being a function of the bending moment as well as of shear, may be small in such cases, or may be otherwise provided for. For this reason the permissible limit of stress for punching shear may be higher than the allowable limit when the shearing stress is used as a means of comparing diagonal tensile stress. The working values recommended are given in Chapter VIII, Section 5.

9. COLUMNS.

By columns are meant compression members of which the ratio of unsupported length to least width exceeds about four, and which are provided with reinforcement of one of the forms hereafter described.

It is recommended that the ratio of unsupported length of column to its least width be limited to 15.

The effective area of hooped columns or columns reinforced with structural shapes shall be taken as the area within the circle enclosing the spiral or the polygon enclosing the structural shapes.

Columns may be reinforced by longitudinal bars; by bands, hoops, or spirals, together with longitudinal bars; or by structural forms which are sufficiently rigid to have value in themselves as columns. The general effect of closely spaced hooping is to greatly increase the toughness of the column and to add to its ultimate strength, but hooping has little effect on its behavior within the limit of elasticity. It thus renders the concrete a safer and more reliable material, and should permit the use of a somewhat higher working stress. The beneficial effects of toughening are adequately provided by a moderate amount of hooping, a larger amount serving mainly to increase the ultimate strength and the deformation possible before ultimate failure.

Composite columns of structural steel and concrete, in which the steel forms a column by itself, should be designed with caution. To classify this type as a concrete column reinforced with structural

steel is hardly permissible, as the steel generally will take the greater part of the load. When this type of column is used, the concrete should not be relied upon to tie the steel units together nor to transmit stresses from one unit to another. The units should be adequately tied together by tie-plates or lattice bars, which, together with other details, such as splices, etc., should be designed in conformity with standard practice for structural steel. The concrete may exert a beneficial effect in restraining the steel from lateral deflection and also in increasing the carrying capacity of the column. The proportion of load to be carried by the concrete will depend on the form of the column and the method of construction. Generally, for high percentages of steel, the concrete will develop relatively low unit stresses, and caution should be used in placing dependence on the concrete.

The following recommendations are made for the relative working stresses in the concrete for the several types of columns:

(*a*) Columns with longitudinal reinforcement to the extent of not less than 1% and not more than 4%, and with lateral ties of not less than $\frac{1}{4}$ in. in diameter, 12 in. apart, nor more than 16 diameters of the longitudinal bar: the unit stress recommended for axial compression, on concrete piers having a length not more than four diameters, in Chapter VIII, Section 3.

(*b*) Columns reinforced with not less than 1% and not more than 4% of longitudinal bars and with circular hoops or spirals not less than 1% of the volume of the concrete and as hereinafter specified: a unit stress 55% higher than given for (*a*), provided the ratio of unsupported length of column to diameter of the hooped core is not more than 10.

The foregoing recommendations are based on the following conditions:

It is recommended that the minimum size of columns to which the working stresses may be applied be 12 in., out to out.

In all cases longitudinal reinforcement is assumed to carry its proportion of stress in accordance with Section 3 (*c*) 6 of this chapter. The hoops or bands are not to be counted on directly as adding to the strength of the column.

Longitudinal reinforcement bars should be maintained straight, and shall have sufficient lateral support to be securely held in place until the concrete has set.

Where hooping is used, the total amount of such reinforcement shall be not less than 1% of the volume of the column, enclosed. The clear spacing of such hooping shall be not greater than one-sixth the diameter of the enclosed column, and preferably not greater than one-

tenth, and in no case more than 2½ in. Hooping is to be circular and the ends of bands must be united in such a way as to develop their full strength. Adequate means must be provided to hold bands or hoops in place so as to form a column, the core of which shall be straight and well centered. The strength of hooped columns depends very much upon the ratio of length to diameter of hooped core, and the strength due to hooping decreases rapidly as this ratio increases beyond five. The working stresses recommended are for hooped columns with a length of not more than ten diameters of the hooped core. The Committee has no recommendation to make for a formula for working stresses for columns longer than ten diameters.

Bending stresses due to eccentric loads, such as unequal spans of beams, and to lateral forces, must be provided for by increasing the section until the maximum stress does not exceed the values above specified. Where tension is possible in the longitudinal bars of the column, adequate connection between the ends of the bars must be provided to take this tension.

10. REINFORCING FOR SHRINKAGE AND TEMPERATURE STRESSES.

When areas of concrete too large to expand and contract freely as a whole are exposed to atmospheric conditions, the changes of form due to shrinkage and to action of temperature are such that cracks may occur in the mass unless precautions are taken to distribute the stresses so as to prevent the cracks altogether or to render them very small. The distance apart of the cracks, and consequently their size, will be directly proportional to the diameter of the reinforcement and to the tensile strength of the concrete, and inversely proportional to the percentage of reinforcement and also to its bond resistance per unit of surface area. To be most effective, therefore, reinforcement (in amount generally not less than one-third of 1% of the gross area) of a form which will develop a high bond resistance should be placed near the exposed surface and be well distributed. Where openings occur the area of cross-section of the reinforcement should not be reduced. The allowable size and spacing of cracks depends on various considerations, such as the necessity for water-tightness, the importance of appearance of the surface, and the atmospheric changes.

The tendency of concrete to shrink makes it necessary, except where expansion is provided for, to thoroughly connect the component parts of the frame of articulated structures, such as floor and wall members in buildings, by the use of suitable reinforcing material. The amount of reinforcement for such connection should bear some relation to the size of the members connected, larger and heavier members requiring stronger connections. The reinforcing bars should be extended beyond the critical section far enough, or should be sufficiently anchored to develop their full tensile strength.

11. FLAT SLAB.

The continuous flat slab reinforced in two or more directions and built monolithically with the supporting columns (without beams or girders) is a type of construction which is now extensively used and which has recognized advantages for certain types of structures as, for example, warehouses in which large, open floor space is desired. In its construction, there is excellent opportunity for inspecting the position of the reinforcement. The conditions attending depositing and placing of concrete are favorable to securing uniformity and soundness in the concrete. The recommendations in the following paragraphs relate to flat slabs extending over several rows of panels in each direction. Necessarily the treatment is more or less empirical.

The coefficients and moments given relate to uniformly distributed loads.

(*a*) *Column Capital.*—It is usual in flat slab construction to enlarge the supporting columns at their top, thus forming column capitals. The size and shape of the column capital affect the strength of the structure in several ways. The moment of the external forces which the slab is called upon to resist is dependent upon the size of the capital; the section of the slab immediately above the upper periphery of the capital carries the highest amount of punching shear; and the bending moment developed in the column by an eccentric or unbalanced loading of the slab is greatest at the under surface of the slab. Generally, the horizontal section of the column capital should be round or square with rounded corners. In oblong panels the section may be oval or oblong, with dimensions proportional to the panel dimensions. For computation purposes, the diameter of the column capital will be considered to be measured where its vertical thickness is at least 1½ in., provided the slope of the capital below this point nowhere makes an angle with the vertical of more than 45 degrees. In case a cap is placed above the column capital, the part of this cap within a cone made by extending the lines of the column capital upward at the slope of 45° to the bottom of the slab or dropped panel may be considered as part of the column capital in determining the diameter for design purposes. Without attempting to limit the size of the column capital for special cases, it is recommended that the diameter of the column capital (or its dimension parallel to the edge of the panel) generally be made not less than one-fifth of the dimension of the panel from center to center of adjacent columns. A diameter equal to 0.225 of the panel length has been used quite widely and acceptably. For heavy loads or large panels, especial attention should be given to designing and reinforcing the column capital with respect to compressive stresses and bending moments. In the case of heavy loads or large panels, and where the conditions of the panel loading

or variations in panel length or other conditions cause high bending stresses in the column, and also for column capitals smaller than the size herein recommended, especial attention should be given to designing and reinforcing the column capital with respect to compression and to rigidity of connection to floor-slab.

(*b*) *Dropped Panel.*—In one type of construction the slab is thickened throughout an area surrounding the column capital. The square or oblong of thickened slab thus formed is called a dropped panel or a drop. The thickness and the width of the dropped panel may be governed by the amount of resisting moment to be provided (the compressive stress in the concrete being dependent upon both thickness and width), or its thickness may be governed by the resistance to shear required at the edge of the column capital and its width by the allowable compressive stresses and shearing stresses in the thinner portion of the slab adjacent to the dropped panel. Generally, however, it is recommended that the width of the dropped panel be at least four-tenths of the corresponding side of the panel as measured from center to center of columns, and that the offset in thickness be not more than five-tenths of the thickness of the slab outside the dropped panel.

(*c*) *Slab Thickness.*—In the design of a slab, the resistance to bending and to shearing forces will largely govern the thickness, and, in the case of large panels with light loads, resistance to deflection may be a controlling factor. The following formulas for minimum thicknesses are recommended as general rules of design when the diameter of the column capital is not less than one-fifth of the dimension of the panel from center to center of adjacent columns, the larger dimension being used in the case of oblong panels. For notation, let

t = total thickness of slab, in inches;
L = panel length, in feet;
w = sum of live load and dead load, in pounds per square foot.

Then, for a slab without dropped panels,

$$\text{minimum } t = 0.024\, L\, \sqrt{w} + 1\tfrac{1}{2};$$

for a slab with dropped panels,

$$\text{minimum } t = 0.02\, L\, \sqrt{w} + 1;$$

for a dropped panel whose width is four-tenths of the panel length,

$$\text{minimum } t = 0.03\, L\, \sqrt{w} + 1\tfrac{1}{2}.$$

In no case should the slab thickness be made less than 6 in., nor should the thickness of a floor-slab be made less than one-thirty-second of the panel length, nor the thickness of a roof slab less than one-fortieth of the panel length.

(*d*) *Bending and Resisting Moments in Slabs.*—If a vertical section of a slab be taken across a panel along a line midway between columns, and if another section be taken along an edge of the panel parallel to the first section, but skirting the part of the periphery of the column capitals at the two corners of the panels, the moment of the couple formed by the external load on the half panel, exclusive of that over the column capital (sum of dead and live loads) and the resultant of the external shear or reaction at the support at the two column capitals (see Fig. 1), may be found by ordinary static analysis. It will be noted that the edges of the area here considered are along lines of zero shear, except around the column capitals. This moment of the external forces acting on the half panel will be resisted by the numerical sum of (*a*) the moment of the internal stresses at the section of the panel midway between columns (positive resisting moment)

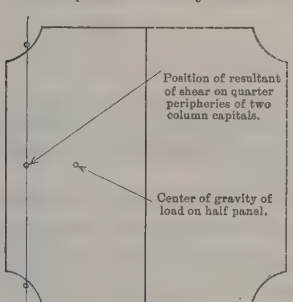

FIG. 1.

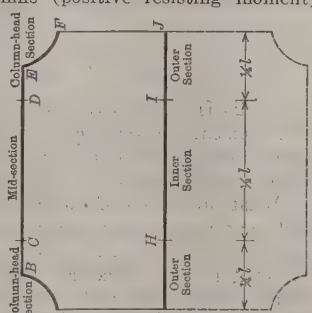

FIG. 2.

and (*b*) the moment of the internal stresses at the section referred to at the end of the panel (negative resisting moment). In the curved portion of the end section (that skirting the column), the stresses considered are the components which act parallel to the normal stresses on the straight portion of the section. Analysis shows that, for a uniformly distributed load, and round columns, and square panels, the numerical sum of the positive moment and the negative moment at the two sections named is given quite closely by the equation

$$M_x = \frac{1}{8}\, wl \left(l - \frac{2}{3}\, c\right)^2.$$

In this formula and in those which follow relating to oblong panels,

w = Sum of the live and dead loads per unit of area;

l = Side of a square panel measured from center to center of columns;

l_1 = One side of the oblong panel measured from center to center of columns;

l_2 = Other side of oblong panel measured in the same way;
c = Diamefer of the column capital;
M_x = Numerical sum of positive moment and negative moment in one direction;
M_y = Numerical sum of positive moment and negative moment in the other direction.

(See paper and closure, "Statical Limitations upon the Steel Requirement in Reinforced Concrete Flat Slab Floors", by John R. Nichols, Jun. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXVII.)

For oblong panels, the equation for the numerical sum of the positive moment and the negative moment at the two sections named becomes

$$M_x = \frac{1}{8} wl_2 \left(l_1 - \frac{2}{3} c\right)^2$$

$$M_y = \frac{1}{8} wl_1 \left(l_2 - \frac{2}{3} c\right)^2$$

where M_x is the numerical sum of the positive moment and the negative moment for the sections parallel to the dimension, l_2, and M_y is the numerical sum of the positive moment and the negative moment for the sections parallel to the dimension, l_1.

What proportion of the total resistance exists as positive moment and what as negative moment is not readily determined. The amount of the positive moment and that of the negative moment may be expected to vary somewhat with the design of the slab. It seems proper, however, to make the division of total resisting moment in the ratio of three-eighths for the positive moment to five-eighths for the negative moment.

With reference to variations in stress along the sections, it is evident from conditions of flexure that the resisting moment is not distributed uniformly along either the section of positive moment or that of negative moment. As the law of the distribution is not known definitely, it will be necessary to make an empirical apportionment along the sections; and it will be considered sufficiently accurate generally to divide the sections into two parts and to use an average value over each part of the panel section.

The relatively large breadth of structure in a flat slab makes the effect of local variations in the concrete less than would be the case for narrow members like beams. The tensile resistance of the concrete is less affected by cracks. Measurements of deformations in buildings under heavy load indicate the presence of considerable tensile resistance in the concrete, and the presence of this tensile resistance acts to decrease the intensity of the compressive stresses. It is believed that the use of moment coefficients somewhat less than those given in a preceding

paragraph as derived by analysis is warranted, the calculations of resisting moment and stresses in concrete and reinforcement being made according to the assumptions specified in this report and no change being made in the values of the working stresses ordinarily used. Accordingly, the values of the moments which are recommended for use are somewhat less than those derived by analysis. The values given may be used when the column capitals are round, oval, square, or oblong.

(*e*) *Names for Moment Sections.*—For convenience, that portion of the section across a panel along a line midway between columns which lies within the middle two quarters of the width of the panel (*HI*, Fig. 2) will be called the inner section, and that portion in the two outer quarters of the width of the panel (*GH* and *IJ*, Fig. 2) will be called the outer sections. Of the section which follows a panel edge from column capital to column capital and which includes the quarter peripheries of the edges of two column capitals, that portion within the middle two quarters of the panel width (*CD*, Fig. 2) will be called the mid-section, and the two remaining portions (*ABC* and *DEF*, Fig. 2), each having a projected width equal to one-fourth of the panel width, will be called the column-head sections.

(*f*) *Positive Moment.*—For a square interior panel, it is recommended that the positive moment for a section in the middle of a panel extending across its width be taken as $\frac{1}{25} wl \left(l - \frac{2}{3} c \right)^2$. Of this moment, at least 25% should be provided for in the inner section; in the two outer sections of the panel at least 55% of the specified moment should be provided for in slabs not having dropped panels, and at least 60% in slabs having dropped panels, except that in calculations to determine necessary thickness of slab away from the dropped panel at least 70% of the positive moment should be considered as acting in the two outer sections.

(*g*) *Negative Moment.*—For a square interior panel, it is recommended that the negative moment for a section which follows a panel edge from column capital to column capital and which includes the quarter peripheries of the edges of the two column capitals (the section altogether forming the projected width of the panel) be taken as $\frac{1}{15} wl \left(l - \frac{2}{3} c \right)^2$. Of this negative moment, at least 20% should be provided for in the mid-section and at least 65% in the two column-head sections of the panel, except that in slabs having dropped panels at least 80% of the specified negative moment should be provided for in the two column-head sections of the panel.

(*h*) *Moments for Oblong Panels.*—When the length of a panel does not exceed the breadth by more than 5%, computation may be made

on the basis of a square panel with sides equal to the mean of the length and the breadth.

When the long side of an interior oblong panel exceeds the short side by more than one-twentieth and by not more than one-third of the short side, it is recommended that the positive moment be taken as $\frac{1}{25} wl_2 \left(l_1 - \frac{2}{3} c\right)^2$ on a section parallel to the dimension, l_2, and $\frac{1}{25} wl_1 \left(l_2 - \frac{2}{3} c\right)^2$ on a section parallel to the dimension, l_1; and that the negative moment be taken as $\frac{1}{15} wl_2 \left(l_1 - \frac{2}{3} c\right)^2$ on a section at the edge of the panel corresponding to the dimension, l_2, and $\frac{1}{15} wl_1 \left(l_2 - \frac{2}{3} c\right)^2$ at a section in the other direction. The limitations of the apportionment of moment between inner section and outer section and between mid-section and column-head sections may be the same as for square panels.

(*i*) *Wall Panels.*—The coefficient of negative moment at the first row of columns away from the wall should be increased 20% over that required for interior panels, and likewise the coefficient of positive moment at the section half way to the wall should be increased by 20 per cent. If girders are not provided along the wall, or the slab does not project as a cantilever beyond the column line, the reinforcement parallel to the wall for the negative moment in the column-head section and for the positive moment in the outer section should be increased by 20 per cent. If the wall is carried by the slab, this concentrated load should be provided for in the design of the slab. The coefficient of negative moments at the wall to take bending in the direction perpendicular to the wall line may be determined by the conditions of restraint and fixedness as found from the relative stiffness of columns and slab, but in no case should it be taken as less than one-half of that for interior panels.

(*j*) *Reinforcement.*—In the calculation of moments, all the reinforcing bars which cross the section under consideration and which fulfill the requirements given under Paragraph (*l*) of this chapter may be used. For a column-head section, reinforcing bars parallel to the straight portion of the section do not contribute to the negative resisting moment for the column-head section in question. In the case of four-way reinforcement, the sectional area of the diagonal bars multiplied by the sine of the angle between the diagonal of the panel and the straight portion of the section under consideration may be taken to act as reinforcement in a rectangular direction.

(*k*) *Point of Inflection.*—For the purpose of making calculations of moments at sections away from the sections of negative moment

and positive moment already specified, the point of inflection on any line parallel to a panel edge may be taken as one-fifth of the clear distance on that line between the two sections of negative moment at the opposite ends of the panel indicated in Paragraph (*e*) of this chapter. For slabs having dropped panels, the coefficient of one-fourth should be used instead of one-fifth.

(*l*) *Arrangement of Reinforcement.*—The design should include adequate provision for securing the reinforcement in place, so as to take, not only the maximum moments, but the moments at intermediate sections. All bars in rectangular bands or diagonal bands should extend on each side of a section of maximum moment, either positive or negative, to points at least twenty diameters beyond the point of inflection, as defined herein, or be hooked or anchored at the point of inflection. In addition to this provision, bars in diagonal bands used as reinforcement for negative moment should extend on each side of a line drawn through the column center at right angles to the direction of the band at least a distance equal to thirty-five one-hundredths of the panel length, and bars in diagonal bands used as reinforcement for positive moment should extend on each side of a diagonal through the center of the panel at least a distance equal to thirty-five one-hundredths of the panel length; and no splice by lapping should be permitted at or near regions of maximum stress, except as just described. Continuity of reinforcing bars is considered to have advantages, and it is recommended that not more than one-third of the reinforcing bars in any direction be made of a length less than the distance center to center of columns in that direction. Continuous bars should not all be bent up at the same point of their length, but the zone in which this bending occurs should extend on each side of the assumed point of inflection, and should cover a width of at least one-fifteenth of the panel length. Mere draping of the bars should not be permitted. In four-way reinforcement, the position of the bars in both diagonal and rectangular directions may be considered in determining whether the width of zone of bending is sufficient.

(*m*) *Reinforcement at Construction Joints.*—It is recommended that at construction joints extra reinforcing bars equal in section to 20% of the amount necessary to meet the requirements for moments at the section where the joint is made be added to the reinforcement, these bars to extend not less than fifty diameters beyond the joint on each side.

(*n*) *Tensile and Compressive Stresses.*—The usual method of calculating the tensile and compressive stresses in the concrete and in the reinforcement, based on the assumptions for internal stresses given in this chapter, should be followed. In the case of the dropped panel, the section of the slab and dropped panel may be considered

to act integrally for a width equal to the width of the column-head section.

(*o*) *Provision for Diagonal Tension and Shear.*—In calculations for the shearing stress which is to be used as the means of measuring the resistance to diagonal tension stress, it is recommended that the total vertical shear on two column-head sections constituting a width equal to one-half the lateral dimension of the panel, for use in the formula for determining critical shearing stresses, be considered to be one-fourth of the total dead and live loads on a panel for a slab of uniform thickness, and to be three-tenths of the sum of the dead and live loads on a panel for a slab with dropped panels. The formula for shearing unit stress given in the Appendix to this report may then be written $v = \frac{0.25\ W}{b\,j\,d}$ for slabs of uniform thickness, and $v = \frac{0.30\ W}{b\,j\,d}$ for slabs with dropped panels, where W is the sum of the dead and live loads on a panel, b is half the lateral dimension of the panel measured from center to center of columns, and jd is the lever arm of the resisting couple at the section.

The calculation of what is commonly called punching shear may be made on the assumption of a uniform distribution over the section of the slab around the periphery of the column capital and also of a uniform distribution over the section of the slab around the periphery of the dropped panel, using in each case an amount of vertical shear greater by 25% than the total vertical shear on the section under consideration.

The values of working stresses should be those recommended for diagonal tension and shear in Chapter VIII, Section 5.

(*p*) *Walls and Openings.*—Girders or beams should be constructed to carry walls and other concentrated loads which are in excess of the working capacity of the slab. Beams should also be provided in case openings in the floor reduce the working strength of the slab below the required carrying capacity.

(*q*) *Unusual Panels.*—The coefficients, apportionments, and thicknesses recommended are for slabs which have several rows of panels in each direction, and in which the size of the panels is approximately the same. For structures having a width of one, two, or three panels, and also for slabs having panels of markedly different sizes, an analysis should be made of the moments developed in both slab and columns, and the values given herein modified accordingly. Slabs with paneled ceiling or with depressed paneling in the floor are to be considered as coming under the recommendations herein given.

(*r*) *Bending Moments in Columns.*—Provision should be made in both wall columns and interior columns for the bending moment which will be developed by unequally loaded panels, eccentric loading,

or uneven spacing of columns. The amount of moment to be taken by a column will depend upon the relative stiffness of columns and slab, and computations may be made by rational methods, such as the principle of least work, or of slope and deflection. Generally, the larger part of the unequalized negative moment will be transmitted to the columns, and the column should be designed to resist this bending moment. Especial attention should be given to wall columns and corner columns.

CHAPTER VIII.

Working Stresses.

1. General Assumptions.

The following working stresses are recommended for static loads. Proper allowances for vibration and impact are to be added to live loads where necessary to produce an equivalent static load before applying the unit stresses in proportioning parts.

In selecting the permissible working stress on concrete, the designer should be guided by the working stresses usually allowed for other materials of construction, so that all structures of the same class composed of different materials may have approximately the same degree of safety.

The following recommendations as to allowable stresses are given in the form of percentages of the ultimate strength of the particular concrete which is to be used; this ultimate strength is that developed at an age of 28 days, in cylinders 8 in. in diameter and 16 in. long, of the consistency described in Chapter IV, Section 2 (*d*), made and stored under laboratory conditions. In the absence of definite knowledge in advance of construction as to just what strength may be expected, the Committee submits the following values as those which should be obtained with materials and workmanship in accordance with the recommendations of this report.

Although occasional tests may show higher results than those here given, the Committee recommends that these values should be the maximum used in design.

Table of Compressive Strengths of Different Mixtures of Concrete.
(In Pounds per Square Inch.)

Aggregate	1:3*	1:4½*	1:6*	1:7½*	1:9*
Granite, trap rock	3 300	2 800	2 200	1 800	1 400
Gravel, hard limestone and hard sandstone	3 000	2 500	2 000	1 600	1 300
Soft limestone and sandstone	2 200	1 800	1 500	1 200	1 000
Cinders	800	700	600	500	400

Note.—For variations in the moduli of elasticity see Chapter VIII, Section 8.

* Combined volume fine and coarse aggregate measured separately.

2. BEARING.

When compression is applied to a surface of concrete of at least twice the loaded area, a stress of 35% of the compressive strength may be allowed in the area actually under load.

3. AXIAL COMPRESSION.

For concentric compression on a plain concrete pier, the length of which does not exceed four diameters, or on a column reinforced with longitudinal bars only, the length of which does not exceed twelve diameters, 22.5% of the compressive strength may be allowed.

For other forms of columns, the stresses obtained from the ratios given in Chapter VII, Section 9, may govern.

4. COMPRESSION IN EXTREME FIBER.

The extreme fiber stress of a beam, calculated on the assumption of a constant modulus of elasticity for concrete under working stresses may be allowed to reach 32.5% of the compressive strength. Adjacent to the support of continuous beams, stresses 15% higher may be used.

5. SHEAR AND DIAGONAL TENSION.

In calculations on beams in which the maximum shearing stress in a section is used as the means of measuring the resistance to diagonal tension stress, the following allowable values for the maximum vertical shearing stress in concrete, calculated by the method given in the Appendix, Formula 22, are recommended:

(*a*) For beams with horizontal bars only and without web reinforcement, 2% of the compressive strength.

(*b*) For beams with web reinforcement consisting of vertical stirrups looped about the longitudinal reinforcing bars in the tension side of the beam and spaced horizontally not more than one-half the depth of the beam; or for beams in which longitudinal bars are bent up at an angle of not more than 45° or less than 20° with the axis of the beam, and the points of bending are spaced horizontally not more than three-quarters of the depth of the beam apart, not to exceed 4½% of the compressive strength.

(*c*) For a combination of bent bars and vertical stirrups looped about the reinforcing bars in the tension side of the beam and spaced horizontally not more than one-half of the depth of the beam, 5% of the compressive strength.

(*d*) For beams with web reinforcement (either vertical or inclined) securely attached to the longitudinal bars in the tension side of the beam in such a way as to prevent slipping of bar past the stirrup, and spaced horizontally not more than one-half of the depth of the beam in case of vertical stirrups and not more than three-fourths of the depth of the beam in the case of inclined members,

either with longitudinal bars bent up or not, 6% of the compressive strength.

The web reinforcement in case any is used should be proportioned by using two-thirds of the external vertical shear in Formulas 24 or 25 in Chapter X. The effect of longitudinal bars bent up at an angle of from 20 to 45° with the axis of the beam, may be taken at sections of the beam in which the bent up bars contribute to diagonal tension resistance, as defined under Chapter VII, Section 8, as reducing the shearing stresses to be otherwise provided for. The amount of reduction of the shearing stress by means of bent up bars will depend upon their capacity, but in no case should be taken as greater than 4½% of the compressive strength of the concrete over the effective cross-section of the beam (Formula 22). The limit of tensile stress in the bent up portion of the bar calculated by Formula 25, using in this formula an amount of total shear corresponding to the reduction in shearing stress assumed for the bent up bars, may be taken as specified for the working stress of steel, but in the calculations the stress in the bar due to its part as longitudinal reinforcement of the beam should be considered. The stresses in stirrups and inclined members when combined with bent up bars are to be determined by finding the amount of the total shear which may be allowed by reason of the bent up bars, and subtracting this shear from the total external vertical shear. Two-thirds of the remainder will be the shear to be carried by the stirrups, using Formulas 24 or 25 in the Appendix.

Where punching shear occurs, provided the diagonal tension requirements are met, a shearing stress of 6% of the compressive strength may be allowed.

6. BOND.

The bond stress between concrete and plain reinforcing bars may be assumed at 4% of the compressive strength, or 2% in the case of drawn wire. In the best types of deformed bar, the bond stress may be increased, but not to exceed 5% of the compressive strength of the concrete.

7. REINFORCEMENT.

The tensile or compressive stress in steel should not exceed 16 000 lb. per sq. in.

In structural steel members, the working stresses adopted by the American Railway Engineering Association are recommended.

8. MODULUS OF ELASTICITY.

The value of the modulus of elasticity of concrete has a wide range, depending on the materials used, the age, the range of stresses between which it is considered, as well as other conditions. It is recommended that, in computations for the position of the neutral

axis, and for the resisting moment of beams, and for compression of concrete in columns, it be assumed as:

(*a*) One-fortieth that of steel, when the strength of the concrete is taken as not more than 800 lb. per sq. in.

(*b*) One-fifteenth that of steel, when the strength of the concrete is taken as greater than 800 lb. per sq. in. and less than 2 200 lb. per sq. in.

(*c*) One-twelfth that of steel, when the strength of the concrete is taken as greater than 2 200 lb. per sq. in. and less than 2 900 lb. per sq. in., and

(*d*) One-tenth that of steel, when the strength of the concrete is taken as greater than 2 900 lb. per sq. in.

Although not rigorously accurate, these assumptions will give safe results. For the deflection of beams which are free to move longitudinally at the supports, in using formulas for deflection which do not take into account the tensile strength developed in the concrete, a modulus of one-eighth of that of steel is recommended.

CHAPTER IX.

Conclusion.

In the preparation of this Final Report, 21 members have taken a more or less active part; all members have agreed to it in its present form.

The Joint Committee acknowledges its indebtedness to its subcommittee on design, Professors Talbot, Hatt, and Turneaure, for their invaluable and devoted service.

The Joint Committee believes that there is a great advantage in the co-operation of the representatives of different technical societies, and trusts that a similar combination of effort may be possible, some time in the future, to review the work done by the present Committee, and to embody the additional knowledge which will certainly be obtained from further experimentation and practical experience with this important material of construction.

Respectfully submitted,

Joseph R. Worcester, *Chairman.*	Arthur N. Talbot,
	William B. Fuller,
Emil Swensson, *Vice-Chairman.*	Edward E. Hughes,
	Albert L. Johnson,
Richard L. Humphrey, *Secretary.*	Gaetano Lanza,
	Leon S. Moisseiff,
John E. Greiner,	Henry H. Quimby,
William K. Hatt,	Sanford E. Thompson,
Olaf Hoff,	Frederick E. Turneaure,
Robert W. Lesley,	Samuel Tobias Wagner,

GEORGE S. WEBSTER,
H. A. CASSIL,
FREDERICK E. SCHALL,
FREDERICK P. SISSON,
JOSEPH J. YATES,
NORMAN D. FRASER,
ROBERT E. GRIFFITH,
SPENCER B. NEWBERRY,
EDWARD GODFREY,*
EGBERT J. MOORE,
LEONARD C. WASON.

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* Mr. Godfrey dissents from the portion of the Report relating to stirrups and their treatment. He would give stirrups and short shear members no recognition, for the reason that he holds that they have not shown themselves to have any definite value in tests, and that analysis fails to show that any definite value can be ascribed to them; he also believes that dependence on stirrups to take end shear has resulted in much unsafe construction and some failures. He would take care of diagonal tension by bending up some of the main reinforcing rods and anchoring them for their full tensile strength beyond the edge of support. He recommends that bends be made close to the supports for the upper bends and at quarter points for the lower bends in beams carrying uniform load. For girders carrying beams bends should be made under the beams. For anchorage he recommends that the rod should extend 40 to 50 diameters beyond the point where it intersects a line drawn, at 45° with the horizontal, from the bottom of the beam at the face of the support.

He recommends that the stress in bent-up rods be assumed to be that obtained by multiplying the excess of shear over that taken by the concrete (at 40 or 50 lb. per sq. in.) by the secant of the inclination of the rod with the vertical.

Mr. Godfrey also dissents from all parts of the Report relating to rodded columns, or columns having longitudinal rods without close-spaced hooping, for the reason that he holds that such reinforcement has not shown itself to have any definite value in tests on columns, and that analysis fails to show that any definite value can be ascribed to it, when such analysis takes into account the necessity for toughness in all columns; he also believes that dependence on such reinforcement has led to much unsafe construction and many failures. He would recognize as reinforced concrete columns only such columns as have in addition to the longitudinal rods a complete system of close-spaced hooping. He objects to the reading of Chapter VII, Section 9, Paragraph (*b*) as being capable of interpretation that hooped columns are given an advantage in the matter of unit stresses only below ten diameters in height. He recommends the standardization of hooped columns, and suggests that columns be reinforced by a coil or hoops of round steel having a diameter one-fortieth of that of the external diameter of the column and eight upright rods wired to the same, the pitch of the coil being one-eighth of the column diameter. He would consider available for resisting compressive stress, the entire area of the concrete of a circular column or of an octagonal column, but no part of the longitudinal rods or hooping. In a square column only 83% of the area of concrete would be considered available. The compression he would recommend on columns (for 2 000-lb. concrete) would be:

$$P = 670 - 12\ \frac{l}{d}$$

where P = allowable compression, in pounds per square inch;
l = length of column, in inches;
d = diameter of column, in inches.

CHAPTER X.

Appendix.

SUGGESTED FORMULAS FOR REINFORCED CONCRETE CONSTRUCTION.

These formulas are based on the assumptions and principles given in the chapter on design.

1. Standard Notation.

(a) Rectangular Beams.

The following notation is recommended:

f_s = tensile unit stress in steel;
f_c = compressive unit stress in concrete;
E_s = modulus of elasticity of steel;
E_c = modulus of elasticity of concrete;
$n = \frac{E_s}{E_c}$;
M = moment of resistance, or bending moment in general;
A_s = steel area;
b = breadth of beam;
d = depth of beam to center of steel;
k = ratio of depth of neutral axis to depth, d;
z = depth below top to resultant of the compressive stresses;
j = ratio of lever arm of resisting couple to depth, d;
$jd = d - z$ = arm of resisting couple;
p = steel ratio $= \frac{A}{bd}$.

(b) T-Beams.

b = width of flange;
b' = width of stem;
t = thickness of flange.

(c) Beams Reinforced for Compression.

A' = area of compressive steel;
p' = steel ratio for compressive steel;
f_s' = compressive unit stress in steel;
C = total compressive stress in concrete;
C' = total compressive stress in steel;
d' = depth of center of compressive steel;
z = depth to resultant of C and C'.

(*d*) *Shear, Bond and Web Reinforcement.*

V = total shear;
V' = total shear producing stress in reinforcement;
v = shearing unit stress;
u = bond stress per unit area of bar;
o = circumference or perimeter of bar;
Σo = sum of the perimeters of all bars;
T = total stress in single reinforcing member;
s = horizontal spacing of reinforcing members.

(*e*) *Columns.*

A = total net area;
A_s = area of longitudinal steel;
A_c = area of concrete;
P = total safe load.

2. Formulas.

(*a*) *Rectangular Beams.*

Position of neutral axis,

$$k = \sqrt{2pn + (pn)^2} - pn \quad \ldots\ldots(1)$$

Arm of resisting couple,

$$j = 1 - \frac{1}{3}k \quad \ldots\ldots(2)$$

[For $f_s = 15\,000$ to $16\,000$ and $f_c = 600$ to 650, j may be taken at $\frac{7}{8}$.]

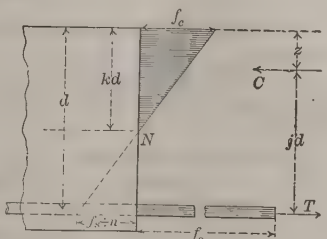

Fig. 3.

Fiber stresses,

$$f_s = \frac{M}{A_s jd} = \frac{M}{pjbd^2} \quad \ldots\ldots(3)$$

$$f_c = \frac{2M}{jkbd^2} = \frac{2pf_s}{k} \quad \ldots\ldots(4)$$

Steel ratio, for balanced reinforcement,

$$p = \frac{1}{2} \frac{1}{\frac{f_s}{f_c}\left(\frac{f_s}{nf_c} + 1\right)} \quad \ldots\ldots(5)$$

(*b*) T-*Beams.*

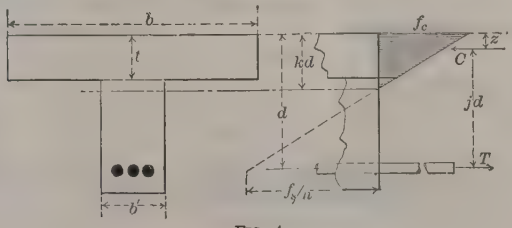

Fig. 4.

Case I. When the neutral axis lies in the flange, use the formulas for rectangular beams.

Case II. When the neutral axis lies in the stem.

The following formulas neglect the compression in the stem.

Position of neutral axis,

$$kd = \frac{2\,ndA_s + bt^2}{2\,nA_s + 2\,bt} \quad \ldots\ldots(6)$$

Position of resultant compression,

$$z = \frac{3\,kd - 2\,t}{2\,kd - t} \cdot \frac{t}{3} \quad \ldots\ldots(7)$$

Arm of resisting couple,

$$jd = d - z \quad \ldots\ldots(8)$$

Fiber stresses,

$$f_s = \frac{M}{A_s jd} \quad \ldots\ldots(9)$$

$$f_c = \frac{Mkd}{bt(kd - \frac{1}{2}t)jd} = \frac{f_s}{n} \cdot \frac{k}{1-k} \quad \ldots\ldots(10)$$

(For approximate results, the formulas for rectangular beams may be used.)

The following formulas take into account the compression in the stem; they are recommended where the flange is small compared with the stem:

Position of neutral axis,

$$kd = \sqrt{\frac{2\,ndA_s + (b-b')\,t^2}{b'} + \left(\frac{nA_s + (b-b')\,t}{b'}\right)^2} - \frac{nA_s + (b-b')\,t}{b'} \quad \ldots\ldots(11)$$

Position of resultant compression,

$$z = \frac{(kdt^2 - \frac{2}{3}t^3)b + [(kd-t)^2\,(t + \frac{1}{3}(kd-t))]b'}{t(2\,kd - t)b + (kd-t)^2\,b'} \quad \ldots\ldots(12)$$

Arm of resisting couple,

$$jd = d - z \quad \ldots\ldots(13)$$

Fiber stresses,

$$f_s = \frac{M}{A_s jd} \quad \ldots\ldots(14)$$

$$f_c = \frac{2\,Mkd}{[(2\,kd - t)bt + (kd-t)^2\,b']jd} \quad \ldots\ldots(15)$$

(c) Beams Reinforced for Compression.

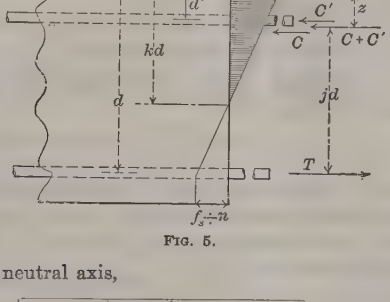

FIG. 5.

Position of neutral axis,

$$k = \sqrt{2\,n\left(p + p'\,\frac{d'}{d}\right) + n^2(p+p')^2} - n(p+p') \quad \ldots\ldots(16)$$

Position of resultant compression,

$$z = \frac{\frac{1}{3}\,k^3 d + 2\,p'nd'\left(k - \frac{d'}{d}\right)}{k^2 + 2\,p'n\left(k - \frac{d'}{d}\right)} \quad \ldots\ldots(17)$$

Arm of resisting couple,

$$jd = d - z \quad \ldots\ldots(18)$$

Fiber stresses,

$$f_c = \frac{6M}{bd^2\left[3\,k - k^2 + \frac{6p'n}{k}\left(k - \frac{d'}{d}\right)\left(1 - \frac{d'}{d}\right)\right]} \quad \ldots\ldots(19)$$

$$f_s = \frac{M}{pjbd^2} = n\,f_c\,\frac{1-k}{k} \quad \ldots\ldots(20)$$

$$f_s' = n\,f_c\,\frac{k - \frac{d'}{d}}{k} \quad \ldots\ldots(21)$$

(d) Shear, Bond, and Web Reinforcement.

For rectangular beams,

$$v = \frac{V}{bjd} \quad \ldots\ldots(22)$$

$$u = \frac{V}{jd\,.\,\Sigma o} \quad \ldots\ldots(23)$$

[For approximate results j may be taken at $\frac{7}{8}$.]

The stresses in web reinforcement may be estimated by means of the following formulas:

Vertical web reinforcement,

$$T = \frac{V's}{jd} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (24)$$

Bars bent up at angles between 20 and 45° with the horizontal and web members inclined at 45°,

$$T = \frac{3}{4}\frac{V's}{jd} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (25)$$

In the text of the report it is recommended that two-thirds of the external vertical shear (total shear) at any section be taken as the amount of total shear producing stress in the web reinforcement. V' therefore equals two-thirds of V.

The same formulas apply to beams reinforced for compression as regards shear and bond stress for tensile steel.

For T-Beams,

$$v = \frac{V}{b'jd} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (26)$$

$$u = \frac{V}{jd \,.\, \Sigma o} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (27)$$

[For approximate results j may be taken at $\frac{7}{8}$.]

(*e*) *Columns.*

Total safe load,

$$P = f_c\,(A_c + nA_s) = f_cA(1 + (n-1)p) \quad \ldots\ldots\ldots (28)$$

Unit stresses,

$$f_c = \frac{P}{A(1 + (n-1)p)} \quad \ldots\ldots\ldots\ldots\ldots\ldots (29)$$

$$f_s = n\,f_c \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (30)$$

DISCUSSION

Mr. Rhett.

A. H. Rhett,* Assoc. M. Am. Soc. C. E. (by letter).—*Integral Water-Proofing of Concrete.*—In this valuable and interesting report the elusory subject of the water-proofing of concrete, or rendering it impervious to the percolation of water, again appears, and again invites discussion.

Reduced to its most practical form, the question becomes merely whether or not concrete can be made non-porous; and, if so, what is the simplest, most effective, and cheapest method of doing it.

The first step in this direction should be, and is, the proper proportioning of the ingredients to fit so closely and compactly that no voids remain, and much experimenting and investigation has been done along these lines. Repeated experimenting, however, has seemed to show that, with the best possible grading, tamping, and mixing with available aggregates, there still remains from 10 to 15% of voids, and there is also an added percentage due to pocketing of water or air during the mixing and placing process.

It is not essential, of course, to close every void to render the concrete water-proof, as far as percolation through it is concerned, because continuity of the voids will play a considerable part in any percolation; but the rendering of the concrete absolutely non-porous is desirable sometimes for other reasons. The entrance into the pores of water affected with acid or alkaline properties gives rise to disintegration of the concrete, from the reaction of the acid or alkali and the free lime always present, and the preservation of the integrity of the concrete is more important than the actual water-proofing of it. Furthermore, any such reaction would eventually render the concrete pervious, even if it were not so originally.

The problem then, in its final form, becomes one of closing this 10 to 15% of voids that remain after the best proportioning has been obtained and those that occur from the pocketing of water or air in the concrete during the mixing process, or from friction between the particles of the aggregates in placing; and a corrective must be sought in means other than proportioning.

A study of the hydration of Portland cement seems to show that such a correction lies latent in the cement itself, and can be made available under proper treatment.

It has been shown† that the so-called beta-orthosilicate element of Portland cement is very slow to hydrate, and that but little trace of its hydration is found in any cement before 7 days, and in some cases it does not appear until 28 days, but that when this hydration does commence it progresses as long as water is present, and with a great

* New York City.

† *Technologic Paper No. 48*, U. S. Bureau of Standards.

Mr. Rhett.

increase in volume. This increase in the volume of Portland cement when continually mixed with water has been also strikingly shown by Nathan C. Johnson,* Assoc. M. Am. Soc. C. E.

This action of the cement, of course, must occur in concrete where the mixing is thorough and when water is held in the mix long enough to permit it. A further point to note is that chemical action of this sort is retarded by pressure, and would be most active in the direction of least resistance and consequently into the void spaces. It is also an interesting fact that this beta-orthosilicate element hydrates with an amorphous and not a crystalline structure. If the latter were the case, there would be a tendency to disrupt the concrete, but the amorphous texture of the increased volume enables it to adjust itself to the void forms and fill them.

It is a well-known fact in daily practice, and has recently been again noted,† that, with a given mix, the strength, and therefore density, will vary directly with the length of time required to dry out.

It would seem entirely possible, therefore, theoretically to get a non-porous concrete:

(1)—If the aggregates were properly proportioned;
(2)—If the mixing were thorough;
(3)—If the placing and tamping were properly done;
(4)—If the quantity of water used were correct; and
(5)—If the forms were kept on long enough to insure the retention of sufficient water to hydrate the cement thoroughly.

Any practical man knows the difficulty and expense, if not the impossibility, of obtaining these conditions, and it becomes a very practical question as to whether or not they might not be obtained by a less expensive and more automatic or fool-proof method.

Answer to this question has been sought in the so-called integral water-proofing compounds, and they have been produced on several theories.

The simplest was the addition of some very finely particled inert material that would fill the voids. There seems to be no reason to doubt that the addition of such material might, to a certain degree, compensate for a faulty proportion of aggregates, but could scarcely be relied on to fill such voids as are caused by the mechanical pocketing of air and water in the concrete during mixing, or by the friction of the particles in placing.

Another theory has been to mix with the concrete a water-repellent material that will coat the voids and pores and prevent absorption.

* "Materials *vs.* Methods—Testimony of Moving Pictures in the Study of Concrete," *Engineering Record*, December 4th, 1915.

† "The Strength and Other Properties of Concrete as Affected by Materials and Methods of Preparation," *Technologic Paper No. 58*, U. S. Bureau of Standards.

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It is difficult to see, though, how such a material could coat the pores without coating also the particles of cement and preventing proper hydration, or how it could be relied on to coat every pore.

A better and more rational theory, however, has been to add a material which will lubricate the aggregates, and thus enable them to settle in a denser mass by eliminating the friction, and which will also act as an inert void filler. Such a material would eliminate the necessity for much tamping, and would compensate to a certain degree for a faulty proportion of aggregates; but there would probably still remain a certain percentage of voids due to the air and water pockets formed in the mixing. These, it would seem, can really only be eliminated by the swelling action caused by the full hydration of the cement, as described previously.

If, however, a material could be added to the concrete that would not only act as a lubricator and void filler, but, in addition, would tend to hold the water in the concrete and prevent too quick drying, it would certainly seem—on perfectly rational grounds—to compensate to a considerable degree for faulty proportioning, careless placing and tamping, inadequate mixing, and early removal of forms.

Such action, moreover, would be automatic, and might well prove much less expensive and more practical than the conditions necessary to obtain the same result without the addition of the compound.

The object of this discussion has been to show, if possible, that there is a theory which is rational and will explain the success that has attended the use of some of these compounds, in so many instances, and that the practical evidence of their effectiveness cannot be candidly disregarded.

To obtain this success, these materials should be used intelligently. There is nothing miraculous about their action. They cannot compensate for cracking or breaking of the concrete, for very lean mixes, or for an acid or alkaline content in the sand, but they can and do compensate, to a degree which is eminently practical, for faulty proportioning, careless placing and tamping, improper mixing, and too quick removal of the forms.

The Appendix to the report repeats the usual or standard textbook formulas for proportioning concrete beams. It has always seemed to the writer that though these formulas are normal for the analysis of a beam already built, they certainly operate in a reverse and roundabout manner for proportioning a new beam.

One must first find p, a proportion of an unknown area, then K, a proportion of an unknown depth, then j, which is a fraction of the real depth, and then d; and, from d, one then works back to the absolute value of p.

It is perfectly possible to solve a reinforced concrete beam as directly as a steel beam, and from the same basic elements, that is, the

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bending moment and the assumed fiber stresses and moduli of elasticity. The writer has used such formulas for years. The formulas given in the report are derived fundamentally from the summation of the horizontal stresses in the concrete and steel; those used by the writer are derived from the moments of these forces. For instance, taking the first condition, a beam reinforced on the tension side only, and using the notation adopted, except that absolute values will be considered instead of proportional ones; that is. Kd will be represented as y, and pbd as a, and, referring to Fig. 6, we would have:

$$\Sigma \text{ of moments of horizontal stresses} = \frac{1}{3} f_c y^2 + a f_s y_2 = 0$$

$$\Sigma \text{ of horizontal stresses} = \frac{1}{2} y f_c - a f_s = 0$$

and

$$y_2 = \frac{f_s}{n f_c} y,$$

from which would result, if $\frac{f_s}{f_c}$ is expressed as r, and M is the moment per unit of width of beam, say, inch-pounds per inch of width,

$$y = \sqrt{\frac{M}{f_c \left(\frac{1}{3} + \frac{1}{2}\frac{r}{n}\right)}}; \; d = y\left(1 + \frac{r}{n}\right); \; a = \frac{y}{2r};$$

for a set of specified values such as $n = 15$, $f_s = 20\,000$ lb., $f_c = 600$ lb., $r = 33.3$,

$$y = \sqrt{\frac{M}{864}}, d = 3.2\, y, \text{ and } a = \frac{y}{67}.$$

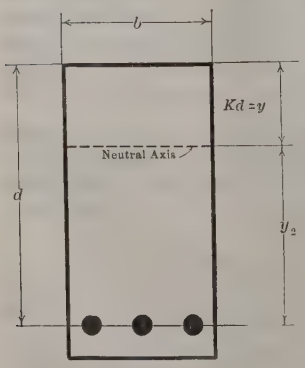

FIG. 6.

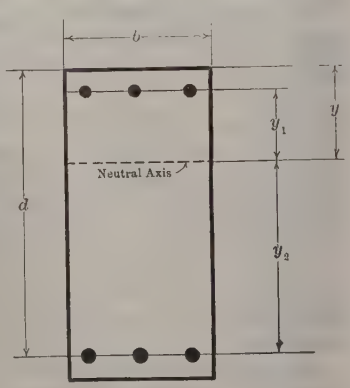

FIG. 7.

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For beams reinforced on both sides, the formulas are not so easily derived, but become equally simple of application for specified values. Two assumptions would have to be made, however: first, the usual one of the ratio of y_1 to y (referring to Fig. 7); and, second, the relative area of a', the area of the steel in compression, to a. The latter assumption would be dictated largely by practical conditions, as in a series of spans of restrained beams. Let $a' = Ca$.

Then,

$$\Sigma M = \frac{1}{3} y^2 f_c + C a f_s' y_1 + y_2 a f_s$$

$$\Sigma H = a f_s - C a f_s' - \frac{1}{2} y$$

which give,

$$M = f_c y^2 \left[\frac{1}{3} + \frac{r}{2n}\left(1 + \frac{C n y_1}{r y}\right)\right]$$

$$y = \sqrt{\frac{M}{f_c\left[\frac{1}{3} + \frac{r}{2n} + \frac{C}{2}\frac{y_1}{y}\right]}}$$

$$d = y\left(1 + \frac{r}{n}\right)$$

$$a = \frac{y}{2r}\left(1 - \frac{C n y_1}{r y}\right)$$

$$a' = C a,$$

which equations become identical with those for single reinforcement when $C = 0$. With specified values, these become equally simple. Using the same notation and with $C = 0.5$ and $\frac{y_1}{y} = 0.75$:

$$y = \sqrt{\frac{M}{973}}$$

$$d = 3.22\, y$$

$$a = \frac{y}{55}$$

$$a' = \frac{y}{27.5}$$

For T-beams reinforced for tension in stem only, and the neutral axis lying within the flange, the forms are the same as for rectangular beams reinforced on one side only, except that M and a represent values per unit width of flange and not of stem.

In the case of the T-beam the axis of which lies in the stem, very simple formulas can be similarly derived with two assumptions which are entirely within the realm of rationality, considering the very radical assumption in T-beam design in regard to the effective width of the flange. These assumptions are, referring to Fig. 8, that the

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averages, $f_c = \frac{1}{2} f_c$ and $Z = \frac{1}{3} t$, are both on the side of safety. Expressing t as Ky, then,

$$\Sigma M = \frac{1}{2} K f_c y \left(y - \frac{1}{3} K y \right) + a f_c y_2$$

$$\Sigma H = \frac{1}{2} K y f_c - a f_s$$

and

$$y = \sqrt{\frac{M}{\frac{1}{2} K f_c \left(1 - \frac{1}{3} K - \frac{r}{n} \right)}}$$

$$d = y \left(1 + \frac{r}{n} \right)$$

$$a = \frac{y}{2 r}$$

$$t = K y$$

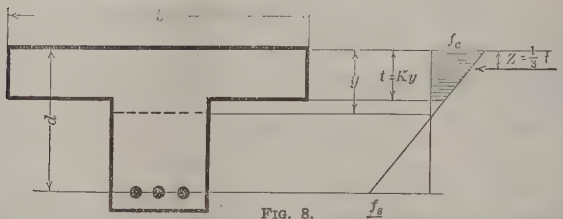

FIG. 8.

TABLE 1.—VALUES OF a FOR ROUND BARS.

DIAMETER, IN INCHES.		⅜	7/16	½	9/16	⅝	11/16	¾	13/16	⅞
Area, a, in square inches.		0.11	0.15	0.196	0.248	0.307	0.371	0.442	0.518	0.601
Values of a spacing.	3 in.	0.037	0.050	0.065	0.082	0.102	0.124	0.147	0.173	0.200
	3½ in.	0.032	0.043	0.056	0.071	0.088	0.106	0.125	0.148	0.172
	4 in.	0.027	0.037	0.049	0.062	0.077	0.093	0.110	0.130	0.150

With the same specified unit values as before, and with $K = 0.75$, the formulas reduce to the same simple forms:

$$y = \sqrt{\frac{M}{675}}$$

$$d = 3.22\, y$$

$$a = \frac{y}{67}$$

$$t = 0.75\, y$$

All these formulas can be read directly from a slide-rule, and the actual area and spacing of the bars could be taken from a table such as Table 1, in which the moment is expressed in inch-pounds per inch of width of beam. Mr. Rhett.

William Fry Scott,* Assoc. M. Am. Soc. C. E. (by letter).—There are many reasons why this Society, as a body, should not approve the Final Report of this Committee as representing an expression of "the present state of the art." One reason will be sufficient to prove this statement, and may be understood by a study of the diagram, Fig. 9. Mr. Scott.

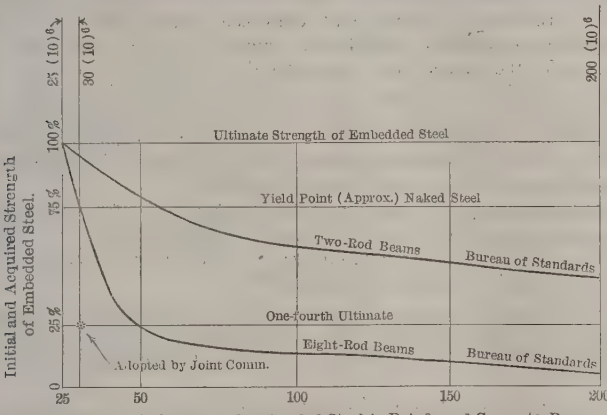

Fig. 9.

This diagram gives the variations in the modulus of elasticity (so-called for want of a better term) of embedded steel, in two of the several groups of reinforced concrete beams made and tested under the direction of Mr. Humphrey, the Secretary of the Committee, and, as computed by the writer on the basis of Assumption *c* 4, page 1125, of the report. On the other hand, the modulus of elasticity of naked steel is assumed, by the Committee, to be identical with that for embedded steel; as is also the case, by the writers of the paper† reporting the beam tests referred to.

This paper has a dual character: On the one hand, it contains the most valuable report of, probably, the most comprehensive series of tests of concrete and reinforced concrete to date. The data are pre-

* Dunville, Ont., Canada.

† Published by the Bureau of Standards, *Technologic Paper No. 2*. "The Strength of Reinforced Concrete Beams; Results of Tests of 333 Beams," By Richard L. Humphrey and Louis H. Losse (June 27th, 1911).

Mr. Scott.

sented in such a thorough manner that they will serve for analysis by a stranger to the experiments just as well as they did for those who planned them. On the other hand, it contains an elaborate thesis that is in no way supported by the data in Tables 3, 33, 34, 35, and 36, and Appendices I and II, of the paper. This thesis follows more or less closely the "design" assumptions on page 1125 of the report of the Committee, and the authors seem to have lost sight of the fact that their real object was to investigate the correctness or falsity of those very assumptions. This thesis shows a tendency to neglect to deal with a long list of partial truths which lead to the whole truth in "the present state of the art" of concrete and reinforced concrete, and its authors missed an opportunity to find clear evidence, in the experiments, to account for the anomaly the Committee devised when "the ratio of the modulus of elasticity of steel to the modulus of elasticity of concrete is taken at 15 [for working stresses, it must be remembered] except as modified in Chapter VIII, Section 8"; and when, "for calculating deflection" for the same working stresses, they are compelled to reduce the ratio by one-half for the same design conditions.

Before proceeding to the actual figures, there are a few suggestions that may possibly clear the way to a correct interpretation of the data to be reviewed. For instance; on page 43 of the paper by the Bureau of Standards, it is stated, on the thesis side:

"Column 12, Tables 6 to 21, contains the unit elongation of the reinforcement at maximum load. This was obtained by dividing the average total deformation of the reinforcement by the gauge length, 29.25 inches."

Now, the gauge length here referred to is that for the reinforced concrete beam under analysis. Then, again, on page 45 of the same paper, there is the statement:

"Given the unit elongation of the reinforcement and assuming a coefficient of elasticity of 30 000 000 the unit stress is easily obtained."

Again, it should be noted that the unit elongation here referred to is that given by the deformations in the plane of the steel in the beam to be analyzed, but the assumed coefficient of 30 000 000 can be none other than that for naked steel.

Immediately following the foregoing quotation the paper states:

"It must be borne in mind, however, that the proportionality of stress and strain holds only within the elastic limit of the material."

Now, a close analysis of the following single example, given by the writer, shows clearly that the modulus of elasticity (so-called) of embedded steel has no relation to that for naked steel, and it also shows that the elastic limit of naked steel bears no relation to that for embedded steel.

Herewith is the example, taken from the paper: Mr. Scott.

It deals with the gravel concrete beams Nos. 342, 343, and 344, one year old. Twenty-one collateral specimens of the concrete used in these beams were tested in cylinders, 8 in. in diameter and 16 in. long, and developed an average unit load of 5 245 lb. per sq. in. at failure—maximum plus 10% and minimum minus 14½ per cent. The same specimens developed an average modulus of elasticity of 5.395 $(10)^6$ lb. per sq. in. with a maximum of 5.86 $(10)^6$ and a minimum value of 4.8 $(10)^6$ lb. per sq. in.

Now, as to the three beams:	Beam No. 342.	Beam No. 343.	Beam No. 344.
Loads given in paper in "analysis of beam at Unit Elongation in Reinforcement Less than that at Yield Point".....	6 000	5 500	6 500
Bending moments, in inch-pounds divided by (bd^2)...	204.5	189.5	219.6
Deformations, upper fiber, in millionths of an inch per inch.	325	282	420
Deformations, steel fiber, in millionths of an inch per inch	872	603	1 111
Neutral axis, distances from top fiber	$0.271d$	$0.319d$	$0.275d$
Now take these data and assume, tentatively, that there is straight-line deformation. Then the stresses on the extreme upper fiber of the concrete, in pounds per square inch, would be.....	1 640	1 320	1 740
and the moduli of elasticity, on the basis of these stresses and the foregoing deformations, would be.....	$5.05(10)^6$	$4.68(10)^6$	$4.15(10)^6$

Thus, since the initial modulus of elasticity found for this same concrete ranged from 5.86 $(10)^6$ to 4.8 $(10)^6$ in standard cylindrical specimens, it must be apparent that straight-line stress was practically obtained on these three beams. In fact, comparing Beam No. 342 with its particular collateral specimen, the beam developed 5.05 $(10)^6$ for a stress of 1 640 lb. per sq. in., or 31½% (ultimate) for cylinders, and the cylinder 5.22 $(10)^6$ at initial stress.

Now, this being the status, for the concrete in compression, it is at once apparent that in the three beams there were stresses of 46 000, 43 300, and 49 500 lb. per sq. in. in the reinforcement, if Assumption *c*4 of the report is correct; and then the modulus of elasticity of the

Mr. Scott.

embedded steel should equal this unit stress divided by the unit deformation recorded in the "log" for the particular condition of loading under consideration, and would be.... and not 30 $(10)^6$, as laid down by the Committee.

	Beam No. 342.	Beam No. 343.	Beam No. 344.
	52.6$(10)^6$	71.2$(10)^6$	44.5$(10)^6$
Then, again (and this is particularly interesting), the breaking loads on these beams, in pounds, would be.	6 280	5 800	6 800
and these exceed that at less than yield point (so-called in the paper) by.....	5% only	6% only	5% only

In other words, it is found that the modulus of elasticity of the embedded steel in these beams ranges from 71 $(10)^6$ to 44½ $(10)^6$ when they are within 6% of their ultimate strength.

Then, again, take the average reading for the three beams at the following applied loads (in pounds).....	2 000	4 000	5 500
Deformations, upper fiber, in millionths of an inch per inch.....	76	146	293
Deformations, steel fiber, in millionths of an inch per inch.....	67	154	650
Neutral axis, distances from top fiber.....	0.53d	0.485d	0.310d
Bending moments, in inch-pounds divided by bd^2.....	84.4	144.4	189.4
From which the stresses on the upper fiber, in pounds per square inch, are.....	387	710	1 360
and these are the following percentages of the average ultimate unit load per square inch on the twenty-one collateral specimens.....	7.4%	13.6%	26%
for which the moduli of elasticity are.....	5.1$(10)^6$	4.85$(10)^6$	4.65$(10)^6$
On the basis of Assumption $c4$ of the report, the stresses on the steel, in pounds per square inch, are.....	20 700	35 200	43 000

	Beam No. 342.	Beam No. 343.	Beam No. 344.
and these stresses divided by the unit deformation (modulus of elasticity) are.....	$310(10)^6$	$230(10)^6$	$66(10)^6$
and, as 6 290 lb. is the mean maximum applied load for the three beams, it is found that for loads at the following percentages of maximum.....	32%	64%	87%
the moduli of elasticity of embedded steel are.....	10 times	7 times	2 times

that laid down by the Committee.

Mr. Scott.

The concrete of these beams was very rich in cement, but exactly similar results can be shown with concrete beams which are poor in cement.

The writer has developed his own figures from the data in the "Log" of the paper mentioned, and they produce entirely different results from those given in the "Summary of Tests", both in the Bureau of Standards paper.

The two curves on Fig. 9 are based on a general analysis of two groups only, namely two-rod beams and eight-rod beams.

Mr. Gayler.

CARL GAYLER,* M. AM. SOC. C. E. (by letter).—The writer gladly acknowledges the great merits of the work of the Committee, and has no doubt that the report will be for a long time an aid to the concrete engineer as a book of reference, guiding him in his work and correcting or sustaining him in his judgment. Aside from a few objections which can be raised, it will probably be accepted as a standard by the Profession. Still, in the writer's opinion, there are such objections, and he begs to state them.

It is to be regretted that so little prominence is given to the important problem of eliminating the dangers from laitance. Every engineer, active in reinforced concrete work, has been aware of this danger for years; many engineers (among them the writer) have come to its realization only after bitter experience; but few have had a clear conception of the magnitude of the problem before reading the classic paper "Water the Chief Factor in the Making of Good Concrete", by Nathan C. Johnson, Assoc. M. Am. Soc. C. E.†

Now, in the Report, Chapter II, Section 2, laitance is not mentioned as one of the causes through which reinforced concrete structures fail, nor is there—and this is more important—any cautioning

* St. Louis, Mo.

† *The Engineering Record*, December 30th, 1916.

Mr. Gayler.

of the inspectors on this point in Section 3 of the same chapter. The "Destructive Agencies" mentioned are: "Corrosion of Metal Reinforcement", "Electrolysis", "Sea Water", "Acids", "Oils", and "Alkalies", but not one word of "Laitance."

The short references in Chapter IV, Section 3, to the removal of laitance can hardly be considered as emphasizing the subject sufficiently, nor is mention made of the chief agency of the trouble, that is, excess of water.

The field covered by reinforced concrete work has grown to such astonishing dimensions that an engineer, no matter what his specialty, to be competent, must have a general knowledge of the principles and requirements of such work. Whether he is engaged in hydraulic works, power stations, bridge or irrigation work, it confronts him in the shape of foundations, conduits, reservoirs, retaining walls, or viaducts. Now, of this immense field, many features of which are still in the midst of evolution, and for which the practising engineer expected enlightenment from the report, not so much on detailed features as through broad, general rules, very little is to be found, with the exception of the indoor slab.

It will be objected that the different types of structures just mentioned do not come within the scope of a report on concrete and reinforced concrete (and, in fact, the Committee makes this claim in the introduction to the report), that such a report is complete after full presentation of the qualities of the materials and of the principles underlying their wise and economical use. Even granting the force of this objection, it is thought that by allowing one-tenth of the space allotted to the flat slab, with its dropped panel, column capital, wall girders, etc., to a few other types of structures of general interest, the value of the report would have been much increased. Some expression of opinion, for instance, on the problems of skeleton retaining walls, on the advantages of the hinged arch, slabs of bridge floors (very different, indeed, with their delicate problems of drainage and shrinkage, from the indoor slab), etc., in the form of concise résumés of the experience of the eminent engineers to whom we owe this report, would have been of great value to the Profession.

Reinforced concrete structures are, essentially, heavy structures. Their success and appearance depend on unyielding support during construction. In many important cases the designing of falsework and centers requires as careful study as the planning of the superstructure; and their maintenance, until the full completion of the work, as much attention as the depositing of the concrete. A reference in the report to this subject, perhaps with the addition of permissible stresses in the materials used, might have been expected. The case is entirely different from the additional types referred to previously. Falsework and centers may be said to form, during construction, an integral part

of the superstructure, and it seems that a general report on concrete and reinforced concrete is not complete if this subject is omitted. Mr. Gayler.

There is one other point on which the writer takes exception to the report, that is, the exclusion of high-grade steel. The Committee has been careful in the wording: it merely recommends the structural grade, but the unit stresses, adopted in the report, practically eliminate the use of high-carbon steel.

High-carbon steel is in general use for reinforced concrete work. It is specified, on even terms, with structural steel, in the standard specifications of the American Society for Testing Materials, of the American Railway Engineering Association, in the building codes of our larger cities, and in the specifications for highway bridge work; and railroad and municipal engineers, as well as engineers in private practice, use it.

As no explanation is given in the report, of the Committee's preference for the milder grade, it may be assumed that the question of greater brittleness of the harder steel has been the deciding one. On this point it is found that Taylor and Thompson, in their treatise on "Concrete, Plain and Reinforced", in a lengthy and clear discussion (p. 414) on the advisability of using hard steel, in which due stress is laid on the difference in the use of steel for bridge work and for reinforced concrete work, and in which careful inspection at the mills is insisted on as an efficient means of obviating dangers from brittleness, end the paragraph with the following (as it seems to the writer) unanswerable argument:

"Steel which can be employed with safety for all locomotive and car wheels of the country certainly cannot be discarded as unsafe for concrete, provided similar precautions are taken in its purchase."

From his own experience, the writer considers high-carbon steel for reinforced concrete work a thoroughly safe material. With proper care and judgment used, where any bending is to be done, no undue risks are run.

Assuming now the safe use of hard steel to be granted (its general use cannot be denied), we reach the important question whether steel with a higher percentage of carbon should be proportioned for the same low unit stress as milder steel (always assuming that the conditions imposed by the bond stress are fulfilled). As far as general practice goes, the answer has been, almost universally, in the negative.

The difference in the unit stresses allowed for high steel over structural steel may be stated, approximately, to be as follows: In railroad work from 2 000 to 3 000 lb.; highway bridge work from 2 000 to 4 000 lb.; and city building laws from 4 000 to 6 000 lb. (the building laws of the City of St. Louis, for instance, specify 14 000 lb. for medium steel and 20 000 lb. for high elastic limit steel).

Mr. Gayler.

Considering that, in the report, not less than fifteen different unit stresses are specified for stone or gravel concretes, according to the nature of the aggregates and the ratio of the mixtures, it seems strange to find the single line "the tensile or compressive stress in steel should not exceed 16 000 lb. per sq. in.", covering the whole subject of proportioning steel, whether the elastic limit is 30 000, 40 000, or 50 000 lb., or the ultimate strength 50 000, 70 000, or 80 000 lb. per sq. in.

Taking into consideration that this question of permissible unit stresses in the steel affects the universal practice in concrete work all over the country, and for every class of such work, there must have been strong reasons for the action of the Committee, and engineers hope to find them stated in the concluding answer to the different discussions on the report.

Mr. Hinckley.

H. V. Hinckley,* M. Am. Soc. C. E. (by letter).—Three points pertaining to "coarse aggregate" occur to the writer as being possibly worthy of mention, namely, large aggregate, glassy aggregate, and strong aggregate.

That "a large aggregate produces a stronger concrete than a fine one" will not be questioned by any competent engineer. It is fortunate, however, that the point has been brought out, because public officials are apt to look on coarse aggregate as a shirking of duty on the part of the rock crusher rather than the exercise of good judgment by the engineer.

The following substitute clause is offered as a suggestion: "The Committee does not feel warranted in recommending the use of blast-furnace slag," flint, or other aggregates which produce smooth, glassy faces, especially for reinforced concrete in which high strains are to be developed.

It is fully as important that the coarse aggregate shall be of tough, strong rock, as it is that "fine aggregate should always be tested for strength." Of two classes of rock (other conditions being equal), the class that, as a beam, will support the heavier load will make, if crushed for coarse aggregate, the stronger concrete. When concrete fails under compression, it fails in tension, and, in failing, it generally splits through the coarse aggregate rather than around it, and as the rock thus broken makes up the greater portion of the section of failure, the strength of the concrete is largely dependent on the strength of the stone. The writer recently stated†:

"Every engineer of experience should know that, under any ordinary concrete specifications, he can build two slabs side by side, using the same sand, the same cement, the same proportion, the same time of mix in the same machine, and at the end of 90 days have one slab at least twice as strong as the other."

* Oklahoma, Okla.

† *Transactions*, Oklahoma Soc. of Engrs., Vol. III.

Mr. Hinckley.

If the writer had to build a reinforced concrete structure, in which high strains were to be developed, he would pay as much attention to the modulus of rupture of the rock to be used as he would to the tests of the fine aggregate. It is time for the Society to get away from the idea that any old stone will make good concrete. A "clean, hard, and durable" stone may be brittle and weak, and it might not be out of place to add, at least, the word "strong."

Since writing the foregoing, the writer's attention has been called to Professor George A. Hool's "Reinforced Concrete Construction", in which is found the only corroboration of the foregoing views which he has ever noticed. Professor Hool says: "Any stone is suitable which is clean and durable, and has sufficient strength to prevent the strength of the concrete from being limited by the strength of the stone," and this means that the stone must be as strong as the mortar.

Mr. Eddy.

HENRY T. EDDY,* ESQ. (by letter).—*1.—The Thickness of Flat Slabs.*—In the first place the writer desires to consider somewhat critically the formula recommended in the report for the value of the minimum total thickness, t, in inches, of a flat slab without dropped panels, in order to show that it is a formula not suited to give the minimum thickness, according to accepted principles in reinforced concrete design. The formula recommended in the report is:

$$t = 0.024\, l\, \sqrt{w} + 1.5 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

in which l = panel length or span between column centers, in feet, and $w = w_0 + w_1$, the sum of the live load, w_0, and the dead load, $w_1 = 12.5\, t$ lb. per sq. ft. Similar formulas have been proposed in the Chicago Ruling† and in the standard building regulations reported to the American Concrete Institute.‡

Equation (1) consists of two parts, the mean effective moment thickness, d, or the depth of the center of the steel below the compressed slab surface, and 1.5 in., which last includes half the thickness of the steel mat of crossed rods plus concrete fire-proofing at least 1 in. thick. Hence, we have the effective moment thickness

$$d = 0.024\, l\, \sqrt{w} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

an equation which, together with Equation (1), the writer has reason to believe is so far from being correct that it should not be recommended or regarded as expressing the minimum value, a conclusion that follows from considerations that will now be given in detail.

The general expression for the bending moment, due to the applied forces acting at any panel-wide vertical section of a square panel of

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† *Engineering News*, September 24th, 1914.

‡ *Proceedings*, Am. Concrete Institute, Vol. XIII, 1917.

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span l ft., when loaded with a total uniformly distributed load of w lb. per sq. ft., or a total load of $W = wl^2$ lb. per panel, may be written in the form:

$$12\ W\frac{l}{a} = 12\ w\frac{l^3}{a}\ \text{in-lb} \qquad (3)$$

in which a is a numerical divisor the value of which has frequently been recommended to be taken at about 24 or 25 for the section at mid-span and at -12 or -15 at the edge of the panel, with intermediate values, positive or negative, at sections situated between these. These, however, are the two sections which are most frequently taken into consideration.

Again, the general expression for the resisting moment of the steel that crosses any vertical cross-section of the panel which has an effective thickness of d in. and a total effective cross-section of steel of $A = 12\ pld$ sq. in., may be written in the form:

$$f_s Ajd = 12 f_s pljd^2\ \text{in-lb} \qquad (4)$$

in which f_s, in pounds, is the unit stress in the steel, jd is the moment arm, $12l$ is the panel length, in inches, and p is the steel ratio of the section.

It is assumed ordinarily that when the direct tensile resistance of the concrete acting in parallel with the tensile resistance of the steel is neglected, the resisting moment must be equal to the applied moment. The writer, however, has reasons, which he has explained elsewhere, for believing that the applied moments are more than twice as large as the actual resisting moments shown by observation. In case this is the fact, it would only be necessary, in order to make Equations (3) and (4) equal, to assume, as the numerically correct value of a, a number which is the same multiple of the value of a, ordinarily assumed, that the applied moments are of the resisting moments. However, leaving this question unsettled for the present, and leaving also the value of a, which will be finally adopted as applicable at mid-span, undetermined, we may nevertheless equate the resisting moment to the applied moment, with the understanding that such a value of a will finally be adopted as may appear to be required by the facts. Hence

$$f_s pjd^2 = w\frac{l^2}{a},$$

or,

$$d = \frac{l\,w}{\sqrt{f_s jpa}} \qquad (5)$$

Now compare Equation (5) for the effective thickness, d, with Equation (2), which was recommended in the report. In case Equation (2) is correct, it must be identical with Equation (5). To express

this relation more concretely, assume that $f_s = 16\,000$ lb. per sq. in., and that $j = 0.9$ at the mid-section. Then, in order that Equations (5) and (2) may be identical, we have Mr. Eddy.

$$16\,000 \times 0.9 \times (0.024)^2 pa = 1, \text{ or } pa = 0.12 \ldots\ldots\ldots (6)$$

Hence, when we take $a = 12, 15, 20, 24, 30, 40, 50, 60,$ we then have $p = 0.01, 0.008, 0.006, 0.005, 0.004, 0.003, 0.0024, 0.002,$ respectively.

It thus appears that in case Equation (2) is, in fact, a correct form of expression for d, then Equation (6) must also hold true; but as a in Equation (6) is a constant which is fixed by the position where the section is taken, at mid-span or elsewhere, and, moreover, is independent of both l and w, it follows from Equation (6) that p is also constant so long as a remains constant. If there is anything in the design of concrete beams, however, that has been indubitably established by experience, it is that the steel ratio increases with the relative thickness, that is, p increases with $\frac{d}{l}$, and the same principle applies to slabs also. Hence, Equations (1) and (2) are not, in fact, correct, and cannot be relied on to fix the minimum value of d, because they require a constant value of p, not in accordance with experience. In fact, p is found to be nearly twice as large in deep slabs for heavy loads as it is in shallow slabs for light loads, in case of well-designed slabs for any given span.

The reason for such increase in the value of p in case of deep beams is evident from the following considerations. In shallow beams and slabs the sharpness of the curvature is inversely proportional to d for a given limiting compression in the concrete at the top surface. The sharper the curvatures the greater the horizontal shearing distortions in the concrete at points distant from the sections where maximum moments occur. Concrete affords a resistance to shearing distortion which is small compared with its resistance to compression. In shallow beams high values of p are futile, for this reason, but, in deep beams, larger values of p may be used than in shallow beams, on account of their smaller shearing distortion. In slabs this still further reason may exist for an increase of p with $\frac{d}{l}$, that is, the crossed rods that make up the mat may properly be more numerous and more closely spaced in a thick slab than in a thin one, thus furnishing more effective co-action of steel and concrete in thick than in thin slabs.

In its present form Equation (2) is not convenient for numerical calculations, because the value of w itself depends on d. It will be

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more convenient to express d as a function of w_0, the live load. Write Equation (2) in the form,

$$d = 0.024\, l\, \sqrt{w_0 + w_1}$$

by the help of the equation

$$w = w_0 + w_1 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (7)$$

and eliminate

$$w_1 = 12.5t = 12.5\ (d + 1.5)$$

by assuming the weight of a cubic foot of concrete to be 150 lb., or 12.5 lb. per in. of thickness per square foot.

Then,

$$d^2 = (0.024\, l)^2\ (w_0 + 12.5\, d + 18.75).$$

Solve this quadratic equation and find,

$$d = 0.00375\ l^2 + 0.024\ l\ \sqrt{(w_0 + 18.75 + 0.0225\ l^2)} \ldots\ldots (8)$$

Table 2 has been computed from Equation (8). Now, if the value of the divisor in Equation (3) is assumed to be $a = 25$ at mid-span of the panel, as recommended in the report, then, by Equation (6), we find the steel ratio, $p = 0.0048$, throughout the entire table, regardless of $\frac{d}{l}$. It is apparent that the thickness found in Table 2 is too great at the larger spans and loads, so much so as almost to prohibit the use of flat slabs for the larger loads and spans figured in the table, but it is evident that this thickness may be reduced by increasing p somewhat with $\frac{d}{l}$.

The effect of this increase of p with $\frac{d}{l}$ will evidently be to produce a proportionate decrease of the dead load and of the thickness of the slab at the larger values of w_0 over that at the smaller values of w_0.

TABLE 2.—Effective Thickness of Flat Slabs.

$d = 0.00375\ l^2 + 0.024\ l\ \sqrt{(w_0 + 18.75 + 0.0225\ l^2)}$ inches; l = span, in feet; w_0 = live load, in pounds per square foot.

Span, in feet.	$w_0 = 100$	200	300	400	500	600
14	4.46	5.75	6.78	7.64	8.42	9.13
16	5.24	6.72	7.87	8.87	9.75	10.56
18	6.05	7.71	9.02	10.13	11.13	12.03
20	6.92	8.74	10.19	11.42	12.52	13.53
22	7.82	9.81	11.40	12.74	13.96	15.06
24	8.76	10.92	12.65	14.12	15.44	16.62
26	9.75	12.08	13.93	15.54	16.95	18.25
28	10.78	13.27	15.26	16.99	18.50	19.89
30	11.86	14.49	16.62	18.45	20.07	21.57
32	12.98	15.77	18.04	19.98	21.71	23.29

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With the view of finding a simple approximate expression for d that will give values of d more nearly in accordance with the results of good practice than those obtained from Equation (4) or from Table 2, it is to be observed that, for any given span, l, Equation (8), regarded as a quadratic expressing the relation between the variables, d and w_0, taken as co-ordinates, represents a parabola. The part of the curve in question ranges from $w_0 = 100$ or less, to $w_0 = 600$ or more. In Fig. 10 the values of d in this range are plotted for $l = 20$ ft. Similar curves occur for other values of l.

Suppose that the effective thickness for two different loads as, for example, at P_1 and P_2 in Fig 10, were sufficiently well ascertained by a consensus of good practice to be supposed to be known, then it would be possible to obtain an equation for a straight line through those two points that would give values of d for those and other loads, which might be more nearly in accord with good practice than those given by the parabola. Such an equation of relation between the effective thickness and the live load has been derived in what follows.

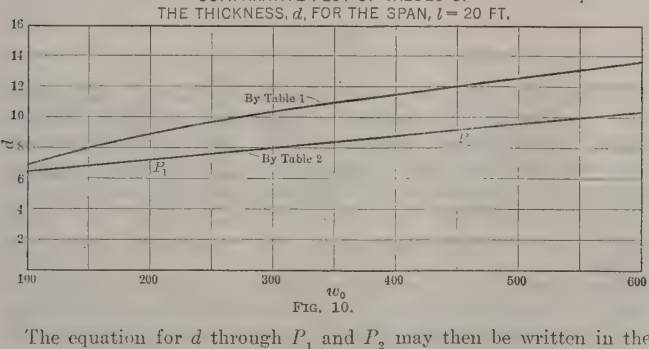

FIG. 10.

The equation for d through P_1 and P_2 may then be written in the form,

$$d = l\,\frac{w_0 + b}{c} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (9)$$

in which l is the span, from center to center of supports, in feet, and w_0 is the total design working load, in pounds per square foot. The total thickness is

$$t = d + 1.5 \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (10)$$

in which, if d is the effective depth of the centers of gravity of the steel below the compression surface of the concrete, 1.5 in. is the remaining part of the total thickness, including one-half of the layer of steel and its fire-proof covering. In the report this is taken as 1.5 in., of which the fire-proofing is required to be 1 in. in the clear.

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In Fig. 10 the two points, P_1 and P_2, at which it was assumed that the two corresponding values of d were known, were both for the same span, l, but the values of the constants, b and c in Equation (9), can be determined equally well in case the known values of d correspond to any different known values of l and w_0 whatever. For example, assume that it is known from experience that, for a 17-ft. panel and a design live load of 200 lb. per sq. ft., the total thickness, $t = 7.6$ in., nearly, and for a 23-ft. panel and a design load of 400 lb. per sq. ft., $t = 11.6$ in., nearly. Substituting these values in Equation (9), gives the following equations from which to determine the values of b and c:

$$\left.\begin{aligned} 7.6 - 1.5 &= 17\ \frac{200 + b}{c} \\ 11.6 - 1.5 &= 23\ \frac{400 + b}{c} \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots (11)$$

Hence, $b = 700$ and $c = 2\,500$, nearly, and Equation (9) becomes

$$\left.\begin{aligned} d &= l\ \frac{w_0 + 700}{2\,500} \\ \text{or,}\quad d &= l\ \frac{4\,w_0 + 2\,800}{10^4} \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots (12)$$

the latter form being more convenient for numerical computation.

In case the examples previously assumed as a basis for the numerical values of b and c used in Equation (12) seem to be unacceptable, then any two other trustworthy and satisfactory numerical examples may be used instead, to arrive at an equation with values of b and c slightly different from those used in Equation (12). Table 3, giving the numerical values of d, has been computed from Equation (12).

The differences in the successive numbers in any row or column of Table 3 are constant, and are noted in the margins at the right and bottom of the table, so as to admit of ready interpolation and exterpolation by proportional parts, which, in this case, is exact and not merely approximate. If, in Equation (12), w_0 and l are regarded as rectangular co-ordinates in a horizontal plane, and d is erected perpendicular to it, the locus of the extremity of d will be a hyperbolic paraboloid which by its height will represent graphically the thickness of the slab. The numbers at the top and left of Table 3 measure such co-ordinates in the plane of the paper, and the numbers in the body of the table express the heights to be laid off at the positions where the numbers are written. The graphical surface thus constructed is a doubly-ruled surface such that a vertical section of it, for any given constant value of either w_0 or l, will be a uniformly sloping straight line.

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The total thickness of the slab will be obtained, as in Equation (10), by adding to d one-half the total thickness of the layers of reinforcing steel plus the thickness of the fire-proofing, which all together may be either more or less than the average value of 1.5 in. recommended in the report.

The values of d in Table 3 are such as to make the thickness, t, throughout the entire table fall within the limit of one thirty-second of the span, as recommended in the report, excepting only the last number at the bottom of the first column, that is, $t = 10.24 + 1.5 = 11.74$, which is $\frac{1}{4}$ in. below the assigned limit.

TABLE 3.—Effective Thickness of Flat Slabs.

$$d = l\,\frac{4\,w_0 + 2\,800}{10^4} \text{ inches;}$$

l = span, in feet; w_0 = live load, in pounds per square foot.

Span, in feet.	$w_0 = 100$	200	300	400	500	600	Δ
14	4.48	5.04	5.60	6.16	6.72	7.28	0.56
16	5.12	5.76	6.40	7.04	7.68	8.32	0.64
18	5.76	6.48	7.20	7.92	8.64	9.36	0.72
20	6.40	7.20	8.00	8.80	9.60	10.40	0.80
22	7.04	7.92	8.80	9.68	10.56	11.44	0.88
24	7.68	8.64	9.60	10.56	11.52	12.48	0.96
26	8.32	9.36	10.40	11.44	12.48	13.52	1.04
28	8.96	10.08	11.20	12.32	13.44	14.56	1.12
30	9.60	10.80	12.00	13.20	14.40	15.60	1.20
32	10.24	11.52	12.80	14.08	15.36	16.64	1.28
Δ =	0.64	0.72	0.80	0.88	0.96	1.04	

It will be noticed that Tables 2 and 3 give practically identical values of $d = 4.5$ in., or $t = 4.5 + 1.5 = 6$ in., at the upper left corner, where $l = 14$ ft., $w_0 = 100$ lb.; and at the upper right and left corners the values of d in Table 2 exceed those in Table 3 by about 25%, and at the lower right corner by nearly 50 per cent. These enormous differences are due to the increase in the value of p with $\frac{d}{l}$ involved in Equation (5), when a is constant.

When the value of $a = 25$ is used, that necessarily implies that the slab has steel enough in one direction to carry the entire load without any stress in the steel at right angles thereto, whereas, in fact, in an inside panel, the steel in both directions acts simultaneously, which requires $a = 50$, and not $a = 25$. This fact will reduce the value of p to one-half that previously found.

The steel ratio derived from Equation (5) is:

$$p = \frac{w\,l^2}{a\,f_s\,i\,d^2} = \frac{(w_0 + 12.5\,d)\,l^2}{16\,000 \times 0.9\,a\,d^2} \qquad (13)$$

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which gives the range of values of p, shown in Table 4. This table gives the numerical value of the ratio of the cross-section of the steel to the concrete at mid-span, in order that, when $a = 25$, the steel will resist the entire applied moment as in a beam, or, when $a = 50$, the steel will resist one-half of that amount, as in a two-way slab. Any intermediate values of l and w_0 may be found correctly from this table by simple interpolation of proportional parts.

TABLE 4.—VALUES OF STEEL RATIO, p, BY EQUATION (13).

	If $a = 25$		If $a = 50$	
	$w_0 = 100$	$w_0 = 600$	$w_0 = 100$	$w_0 = 600$
$l = 14$	0.0047	0.0073	0.0024	0.0036
$l = 32$	0.0067	0.0083	0.0034	0.0042

These values of p show that Table 3 requires no excessive steel ratios, either at mid-span or at panel margins, where, since the values of a will be about one-half those at mid-span, the values of p will be about twice as great.

In obtaining the values of d in Table 3, no allowance has been made for any reduction of span by reason of size of capitals. In order to make such allowance, it will be sufficient to assume that l is the effective span, instead of the actual span, and use it as the tabular span for finding d in Table 3.

It will now be in order to find out, in case slab thicknesses in accordance with Equation (9) and Table 3 are adopted, whether either the compressions in the concrete will be excessive, or the deflections of the slab will be greater than is permissible. For, if neither of these things occurs, then we have established the admissibility of these thicknesses, which are less than those found by Equation (1) and Table 2, and, hence, these latter will not be actual minimum values, but should be replaced by other smaller values found from Equation (9). The compression in the concrete is to be found from the common formula, $f_c = \frac{f_s k}{n\,(1 - k)}$, in which k must be less than 0.4, and probably does not exceed 0.3; and we may assume $f_s = 16\,000$, and $n = 15$, so that the unit stress, f_c, does not exceed 700 lb., and probably is not more than 500 lb., at any point, unless at the edge of the capital, at which point a special investigation may be necessary. This shows that, in general, the compression in the concrete should not be regarded as excessive.

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An expression for the deflection, z, at the panel center of a flat slab has been found to be:

$$z = \frac{W\ L^3}{4.75 \times 10^{10}\ d^2\ A_1} \quad \ldots\ldots\ldots\ldots\ldots\ldots (13)^*$$

in which $L = 12\ l$ is the span, in inches (not feet), and A_1 is the cross-section of a side belt.

Now,

$$A_1 = \frac{1}{2}\ A = \frac{W\ L}{2\ f_s\ a j\ d},$$

which last was obtained by equating applied and resisting moments as in Equations (3) and (4). Substitute this value of A_1 in Equation (13), and assume $f_s = 16\,000$, $a = 25$, $j = 0.9$, and let the slab be as thin as permissible, namely, $\frac{L}{d} = 32$. We then have the largest possible relative deflection for a unit steel stress of 16 000 lb. Then, the relative deflection,

$$\frac{z}{L} = \frac{2\ f_s j\ a\ L}{4.75 \times 10^{10}\ d} = \frac{1}{2\ 060} \quad \ldots\ldots\ldots\ldots\ldots\ldots (15)$$

This relative deflection is so small that it would permit a test load of more than 2.5 times the sum of the design load plus the dead load, without causing $\frac{z}{L}$ to exceed $\frac{1}{800}$, provided the steel were not thereby stressed beyond its elastic limit, which, in this case, would have to be at about 40 000 lb. per sq. in. Smaller values of $\frac{L}{d}$ than 32, such as occur throughout almost the entire Table 3, will give smaller values of $\frac{z}{L}$ than the limiting value computed in Equation (15). On the other hand, although a value of a larger than 25 would have an effect to offset the smaller values of $\frac{L}{d}$ and so to increase $\frac{z}{L}$, yet the observed values of f_s are invariably much smaller than those computed by Equation (14), which serves actually to reduce the value of $\frac{z}{L}$ at least to that found by Equation (15). It follows, consequently, that values of d as small as those in Table 3 may be used with safety, and that values as large as those in Table 2, as prescribed by Equations (1) and (2), are not a minimum, and it has been further shown that they are unnecessarily large.

* See "Concrete Steel Construction", Eddy and Turner, (71), p. 204.

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The report recommends minimum total thicknesses that may be written as follows:

$$\left.\begin{aligned} &\text{For a slab without a drop, } t = 0.024\, l\, \sqrt{w} + 1.5 = d + 1.5 \\ &\text{For slab with drop, } \quad t = 0.02\, l\, \sqrt{w} + 1 = \frac{5}{6}\, d + 1 \\ &\text{For the drop itself, } \quad t = 0.3\, l\, \sqrt{w} + 1.5 = \frac{5}{4}\, d + 1.5 \end{aligned}\right\} \ldots\ldots (16)$$

and, further, that the width of the drop is $0.4\ L$, so that its area will be $(0.4\ L)^2 = \frac{L^2}{6}$, nearly. Hence, the total volume, in cubic inches, of a panel without a drop would be:

$$v = (d + 1.5)\ L^2 \ldots\ldots (17)$$

but the volume of a panel together with a drop would be:

$$v = \left(\frac{5}{6}\, d + 1\right)\frac{5}{6}\, L^2 + \left(\frac{5}{4}\, d + 1.5\right)\frac{L^2}{6}, \text{ nearly, or}$$

$$v = \left(\frac{65}{72}\, d + 1.1\right)\ L^2 \ldots\ldots (18)$$

From this it is evident that the report discriminates against the flat slab without a drop and in favor of the slab with a drop by making the latter so much thinner that the total volume per panel is less. In the writer's opinion such discrimination is without justification in theory or practice. In his opinion, in order to make the formulas for the thicknesses of slabs with a drop comparable with those without a drop, the volume of the former should be at least as great as the latter. With this in view, it will be necessary, in order to have the drop 50% thicker than the rest of the slab and its area $= \frac{L^2}{6}$ to write for the thickness of a slab with a drop,

$$\left.\begin{aligned} &t = 0.022\, l\, \sqrt{w} + 1.5 = \frac{11}{12}\, d + 1.5 \\ &\text{and for the drop itself,} \\ &t = 0.0033\, l\, \sqrt{w} + 1.5 = \frac{11}{8}\, d + 1.5 \end{aligned}\right\} \ldots\ldots (19)$$

Hence,

$$v = \left(\frac{11}{12} d + 1.5\right)\frac{5}{6}\, L^2 + \left(\frac{11}{8}\, d + 1.5\right)\frac{1}{6}\, L^2$$

or, $v = (d + 1.5)\ L^2$, nearly.

The reason for the foregoing statement is this: In the case of a given uniform loading, the numerical sum of the bending moments at mid-span and one margin is constant, but the sub-division of this

constant total moment between mid-span and margin depends on the structure of the slab, as follows: Either the line of inflection in the slab occupies a position fixed by the simultaneous dip of the reinforcing rods as they drop below the neutral surface, or it is undetermined, due to the irregular dipping of the steel, and, in that case, is determined only by the relative stiffness of the columns and of the panel at the margin, as compared with that at mid-span. The report does not contemplate any fixed lines of inflection, for it makes express provision for irregular dipping of steel rods. However, it recommends that steel be provided to resist moments of certain given magnitudes at mid-span and margin separately, regardless of the presence or absence of a drop. If that recommendation is to be adopted, the suspended portion of the panel must needs be just as thick in case of a drop as without it. The only excuse for its being made thinner with a drop would be that the columns and the cantilever portion containing the drop had been made stiffer and the cantilever made larger thereby, so that the lines of inflection were thereby made to occupy positions somewhat more distant from the columns than in the case where there is no drop. This would require the steel over the drop to be kept near the top of the slab for a greater distance than in a plain flat slab, and would also require larger and stiffer columns to resist the unequalized moments, but would hardly permit the total volume of the panel to be decreased, as has been done in the report. Mr. Eddy.

It is still an unsolved problem to determine the economical relative sizes of the suspended portions of the panel and of the cantilever, and the relative stiffness of the columns, as well as the manner of distribution of the resisting moments across the margins of the panels. On the solution of this problem evidently no light has been shed by the recommendations of the report, since it recommends a difference in thickness without any difference in the resisting moment—a recommendation hard, if not impossible, to justify.

Equations (19) for thicknesses of slab and drop to replace those recommended in the report, assume in effect that the cantilever portion of a slab with a drop will be slightly larger than without a drop, but do not proceed to such unjustifiable lengths as the corresponding expressions in Equations (16) given in the report, which wholly ignore the redistribution of the constant total moment between mid-span and margin and the increase of that part of the unequalized moment distributed to any column due to decreasing the stiffness of the panel at mid-span.

In adopting Equations (19) for the thickness of the slab and drop, it is understood that the values of d which are to be inserted, in order to obtain minimum values of t, are those obtained from Equation (12) and Table 3, although the foregoing criticism of Equations (16) is

Mr. Eddy. equally valid were we to adopt as correct the values of d derived from Equation (8) and Table 2.

2.—Flat Slabs as Statically Indeterminate Structures.—In the second place, the writer objects to the adoption in the report of any theory of flat slabs as correct, which assumes them to be statically determinate structures, because they cannot be so regarded. The report, however, on page 1138, refers to the paper of John R. Nichols,* Jun. Am. Soc. C. E., as affording an approximate basis for its formulas. Now, any assumption of the validity and applicability of statical analysis to continuous flat slabs is incorrect, and leads to erroneous results, just as much as in the case of continuous beams or of any other indeterminate structures.

Any structure in which the magnitude or distribution of the stresses in any part of it undergoes any alteration by varying the rigidity of any of its members or elements is statically indeterminate. Hence, the principles of statics cannot be assumed to be applicable to such a structure unless there is definite proof that the statical principles sought to be applied are actually valid for the case in hand, so that it is beside the mark to adduce any statical limitations or requirements in flat slabs, such as are adduced in the report, because statical principles must here be subordinated to the principle of rigidities, which is the guiding principle in all indeterminate structures.

That a flat slab continuous over an unlimited array of separated supports is an indeterminate structure according to the above definition is evident from comparing the distribution of the stresses across the sides of the panels, due to the bending moments in a slab having most of the reinforcing steel concentrated in belts passing over the supports, with the distribution of the stresses across the sides of the panels where this is not the case, as it is not in the crown-sheet of a steam-boiler, for example. For, in the former case, the stresses across the sides due to the bending moments are largely concentrated at and near the supports, but it is far otherwise in the crown-sheet, which is supported on stay-bolts that pass through holes in the sheet, which completely destroy the radial resistance to bending at the supports. By reason of this distribution of the rigidity in the crown-sheet, the entire resistance to bending across any side of a panel is afforded by that part of the side lying between supports, and none of it is at the supports.

Statical principles, therefore, may be applied to flat slabs only after due inquiry and after it has been ascertained to what extent they may be applied to them. Statics and the principle of the lever are so ingrained in all reasoning about structures that the unconscious assumption of their applicability is likely to vitiate otherwise well-considered investigations of structures so complex in their interactions as inde-

* "Statical Limitations upon the Steel Requirement in Reinforced Concrete Flat Slab Floors", *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 1670 (1914).

terminate structures must necessarily be, and great care must be exercised in order to avoid the conclusions that inevitably follow from yielding to the insidious temptation unconsciously to make this assumption. The following unavoidable conclusion, therefore, may be formally stated: Mr. Eddy.

Proposition I.—Continuous flat slab floors on separated supports are indeterminate structures subject to the law of relative rigidities and its corollary, the law of least work.

In any indeterminate structure the relative rigidities of its parts control the distribution of the stresses in it strictly in proportion to the relative rigidities of the paths by which those stresses are carried, so that the greatest total stresses occur wherever the structure offers the greatest total resistance to deformation.

The steel reinforcement in a slab affords the principal resistance to tensile stress, and hence the total tensile resistance is, in general, where the cross-section of steel is greatest. Consequently, the distribution of the tensile stresses will in general be controlled by the quantity and position of the steel in the slab.

3.—Shearing Stresses in Flat Slabs.—Equilibrium requires that the total vertical shearing stress across the perimeter or margin of any panel shall be equal to the sum of the total live load resting on the panel added to the weight of the panel itself. The perimeter of the panel may be taken approximately at the sides of the rectangle the corners of which are at the four adjacent column centers, but it may be found somewhat more exactly by replacing parts of this rectangle near the columns by parts of the perimeters of the column capitals for 90° around each column center.

Shearing stresses transmit loads horizontally by the help of the bending moments which they bring into play, and they re-apply those loads undiminished in total amount at the supports at some distance horizontally from their initial positions.

In order to discuss this matter mathematically, designate the total uniformly distributed load on one panel by W, then, in case of a square panel loaded uniformly and supported at its four sides by walls or equal girders, the total vertical shear across a section close to one side is $\frac{1}{4}W$, as is evident either by symmetry or by the principle of the equal rigidities of the possible paths, lying in two directions at right angles to each other, by which the load may be carried to the supports.

Again, take the case of a single inside panel of any equally loaded panels supported on columns with square capitals, the edges of which are parallel to the sides of the panels; then, on the same principle, it is evident that the vertical shear across each side of each capital is $\frac{1}{4}W$. Now, whatever may be the shape of the perimeter of the capitals, the sections on which vertical shear occurs may be taken to be made up

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of an infinite number of successive elementary vertical faces all of which are parallel either to one or other of the sides of the panel. Then the total shear on all the vertical faces on one side of a capital, which are parallel to one side of a panel, is $\frac{1}{4}W$. This may be regarded as equivalent to saying that, whatever shape the perimeter of the capital may actually have, it can be assumed without material error for the purposes of computation to be replaced by a square capital.

It will be noticed that the intensity of the vertical shear across the perimeter of the panel is distributed in a very different manner, in the case of supporting walls on four sides, from its distribution across the edges of square capitals, in the case of supporting columns, for, in the latter case, the shear is all concentrated at the perimeter of the capitals and is zero at the other parts of the perimeter of the panel between capitals, although the total amount in either case must be equal to the panel load, W.

This concentration of shear is brought about by the bending and deformation of the slab, in which a shallow saucer-shaped hollow is produced around each panel center, while around each column center the slab is bent into the shape of an inverted saucer. The stresses accompanying these deformations effect the distribution of the shears just mentioned, as well as that of the bending moments also, a matter which will be treated later.

In case of wall supports on the sides, slab deformations are very different from this, as they produce the distribution of shears and moments that occur at the walls, which are very different from those at the perimeter of the capitals, a matter that has been treated by the writer elsewhere.* In all cases, the distribution of the vertical shears depends on, and is controlled by, the relative rigidities of the various parts of the perimeter and supports, as expressed by their capacity to resist vertical forces without yielding. Any subsidence or yielding of a support or any part of it will alter the distribution of shears that otherwise would take place and reduce their intensity, and any inequalities in the resistance of the material at the perimeter of a capital will cause corresponding inequalities in distribution of the shears.

Again, consider a single square panel that is one of a single tier of equal square panels which constitute a wide beam and are supported on a succession of equi-distant transverse parallel walls or rigid girders across the tier. In this case no saucer-shaped hollows are produced by the load, for simple flexure alone occurs in such a case, and the deformation is designated as cylindrical, meaning thereby that the surface produced by flexure is a ruled surface such that any sections of it by vertical planes parallel to the wall supports are horizontal straight lines parallel to the walls. In this structure the entire shear-

* "Concrete-Steel Construction", Eddy and Turner, p. 285.

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ing rigidity at the sides of a panel is at the walls, and is zero at the two unsupported sides. The total shear at each supported side is $\frac{1}{2}W$ and the total shear at any parallel vertical section between the walls will be twice that occurring at the corresponding section of an inside panel of a continuous slab of several tiers of panels on separated supports, such as was considered previously. As just stated, this shear of $\frac{1}{2}W$ is due to the fact that there are no supporting resistances or rigidities at the two unsupported sides of this panel.

This fundamental difference in the distribution of the total vertical shear at or across panel-wide sections in those two cases is of the utmost importance in the theory of flat slabs, because, owing to the fact already stated that a given panel load, W, can produce a total shear at the perimeter no greater than W, not only are the total shears in sections parallel to the sides of the panels of flat slabs on separated supports 50% less than in the wide beam structure just described, but, by reason of the necessary relation existing between the shears and the bending moments, the total applied bending moment produced at any panel-wide section of an inside panel is likewise reduced to 50% of that which would be produced if the total load were carried, as in the beam, by moments across that section and sections parallel to it, and none of it was carried by moments across sections at right angles thereto. Now, since in a continuous slab on separated supports equal rigidities occur in two sets of sections at right angles to each other, both must act at the same time, and each does 50% only of that which it would do in the absence of the other. In other words, since the shears in the sections parallel to either one of the sides of the panels in a continuous slab on separated supports are those that arise from a uniform panel load of $\frac{1}{2}W$, and equal shears also occur in sections at right angles thereto, it follows that the applied bending moments across each of the set of panel-wide sections parallel to the sides of such a panel are those arising from a uniformly distributed panel load of only $\frac{1}{2}W$, instead of a load, W, as was the case in a beam.

However, a wall panel in a flat slab floor that rests on a side-wall of the building at one side of the panel is prevented by the wall from having any deflection at that edge, so that the curvature of the panel under load is more nearly cylindrical near the wall than it would be with separated column supports at the edge. This makes the action of the panel more nearly like that of a wide beam with one end resting on the wall. In other words, the total shear in a section close to the wall will be greater than $\frac{1}{2}W$, but less than $\frac{1}{2}W$, while the total shear in a section at right angles to the wall and at the edge of the panel will be less than $\frac{1}{4}W$. The applied bending moments in these two directions in the panel will be such as to correspond to those shears.

An outside panel, however, the outer edge of which is supported on separated columns only, with no additional girder or stiffening

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along its outer edge, undergoes a greater deflection at the outer edge between supports than at other edges, and the total shear at a section near the outer edge is less than at corresponding sections near the other edges of the panel, and needs additional stiffening in the slab, either at the outside edge or parallel thereto. The effect of this kind of column support is to make the total shear in a section near the outer edge of the panel less than $\frac{1}{4}W$, while it makes that in a section of the panel at right angles thereto and near the side of the panel greater than $\frac{1}{4}W$. Corresponding changes occur in the total applied bending moments in the two directions in these panels. This last kind of outside panel with others contiguous thereto takes on more or less of the properties of a beam along the outside of the slab in which it differs materially from an outside panel resting on a wall, which last, it was seen, had some of the properties of a beam with an end at the wall.

A comparison of the two preceding paragraphs shows that outside panels supported at the edge of the slab on an outside row of columns may be designed so that their action will resemble closely that of the inside panels. In order that this may occur, the reinforcement at the outside edge must be such that at the heaviest loads to which it is subjected the deflections will be the same at mid-span between the columns at the edge as occur under these loads at mid-span between interior columns. These identical deformations will ensure nearly equal shears and bending moments in outside and inside panels, because the saucer-shaped deformations will be nearly the same in each.

4.—Applied Bending Moments in Flat Slabs.—It is a well-known proposition of beam theory that the sum total of half the numerical values of the applied negative bending moments at the ends of any span of length, L, plus the numerical value of the positive bending moment applied at mid-span, all arising from a uniformly distributed load, W, amounts to $\frac{WL}{8}$, whether the span is a simple one with end supports merely, or is continuous, or partly so, and this without regard to the loading or absence of loading on other spans of the beam. This proposition holds true regardless of the relative magnitudes of the moments of inertia of the beam at its successive cross-sections by which the relative rigidities of the beam in resisting the applied bending moments is expressed. It likewise is independent of the rigidity or lack of rigidity of the supports or their connections with the beam in resisting applied bending moments. The proposition, therefore, considers such a beam as an indeterminate structure to which the principle of rigidities applies in determining how the total moment, $\frac{WL}{8}$, is distributed between mid-span and ends. It will be convenient to designate this constant quantity, $\frac{WL}{8}$, as the total applied bending moment in the

span, L, due to the uniform load, W. A proof of this well-known proposition may be found, among other places, in a paper by the writer.* Mr. Eddy.

In accordance with the conclusions already reached in this discussion, the total applied bending moment in each direction parallel to the sides in a panel of a continuous slab on separated supports depends on the part of the load that is transmitted by the shears in that direction, so that, in an inside panel of such a slab, only one-half of the bending moment would be applied in each direction, and therefore the sum total of the numerical value of the positive applied bending moment across a mid-section parallel to one side added to the numerical value of the negative applied bending moment across that side is only $\frac{WL}{16}$. In any case, the sum, as above specified, of the applied bending moments in the two spans lying at right angles in any panel would together have the constant value, $\frac{WL}{8}$.

In view of this, and disregarding for the instant the effect of the size of the supports, we will state:

Proposition II.—In any square panel, inside or outside, of a continuous flat slab on rows of separated supports placed at successive distances of L from each other and carrying a total uniformly distributed panel load, W, the sum total of the numerical values of the positive applied bending moments acting across the two panel-wide mid-sections drawn at right angles parallel to the sides, added to half the sum total of the applied bending moments acting across the four sides of the panel, is equal to the constant quantity, $\frac{WL}{8}$.

This proposition may be readily demonstrated for an inside panel in which the rigidities are equal and symmetrical about both of the mid-sections. For,

Let S_x represent the total vertical shear across a panel-wide section at a distance, x, from the panel center;

and let M_x represent the total applied bending moment at the same section.

Then, $dM_x = S_x dx$, for this equation expresses the necessary fundamental relation of shear to applied moment along x.

Similarly, for sections at right angles to this, the corresponding equation may be written:

$$dM_y = S_y dy.$$

* "A Further Discussion of the Steel Stresses in Flat Slab Floors", *Proceedings*, Am. Concrete Institute, 1916, p. 284.

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Now, combining sections at right angles to each other and at equal distances from the panel center at which $x = y$, the sum of the two vertical shears across these sections is equal to the total load between either of them and the mid-section at $x = 0$, or at $y = 0$; that is, when $x = y$, we have $S_x + S_y = \frac{Wx}{L}$, in which $\frac{W}{L}$ is the total panel load per unit of width or length of panel.

In case of equal rigidities in directions at right angles, at equal distances, $x = y$, from mid-section, we have $S_x = S_y$, but not in case of unequal rigidities. In any case, at $x = y$ we have $S_x + S_y = \frac{Wx}{L}$.

Hence, by addition, at $x = y$,

$$d\,(M_x + M_y) = (S_x + S_y)\,d\,x = \frac{W\,x\,d\,x}{L},$$

and integrating between $x = 0$ and $x = \frac{1}{2}\,L$,

$$(M_x + M_y)_{x\ =\ y\ =\ \frac{1}{2}\,L} - (M_x + M_y)_{x\ =\ y\ =\ 0} = \frac{W\,L}{8},$$

that is, the excess of the sum total of the positive moments across the two panel-wide mid-sections drawn at right angles to each other over the sum total of the negative moments across two sides mutually at right angles is $\frac{W\,L}{8}$. This excess, however, is the sum of the numerical values of these moments, which is the result stated in Proposition II. This holds true in general in case of unsymmetrical rigidities, although the demonstration is less simple for the more general case.

The distribution of the bending moment along each panel-wide section depends on the relative resistance to bending of the several parts of that section, as was pointed out, for example, where the bending resistance of the crown-sheet of a boiler was considered. The sub-division of the total applied bending moment, $\frac{W\,L}{8}$, between the two directions at right angles, however, depends not only on their relative resistances to bending moments at these sections, but also on their capacity to resist vertical shears as well.

As a result of the preceding investigation, we have consequently established the following:

Corollary.—In a square interior panel under a uniform load, W, the total numerical value of the positive bending moment at a mid-section, added to the total numerical value of the negative bending moment at an edge parallel to the mid-section, is only one-half of $\frac{W\,L}{8}$.

Disregarding for the instant the reduction of span due to the size of the capital, the report says: Mr. Eddy.

"Analysis shows that * * * the numerical sum of the positive moment and the negative moment at the two sections named is given quite closely"

by $\frac{WL}{8}$, when written in the notation used herein. This shows that, for an inside panel, the analysis adopted in the report would make the applied moments twice as great as has just been shown really to exist. It is stated in the report, however, that the use of moment coefficients somewhat less than those derived by this analysis is believed to be warranted, and the reason assigned for this is the tensile resistance of the concrete. How small the allowance to be made on this account is, may be estimated from the recommended coefficients of $\frac{1}{25}$ at mid-span and $\frac{1}{15}$ at a margin, the sum of which is practically $\frac{1}{9.4}$ in place of $\frac{1}{8}$, as given by analysis. It is clear, therefore, that the recommendations in the report practically require provision to be made, so far as resisting applied moments are concerned, sufficient to carry the entire load by resistance in one direction only and then require, in addition, that provision shall also be made to carry the entire load by resistance in a section direction at right angles to the first, when, in fact, one-half must be carried in each direction. It is perhaps unnecessary to remark that since moment magnitudes are of the nature of directed quantities, moments at right angles to each other are mutually independent of each other.

5.—Lines of Inflection.—As ordinarily understood, lines of inflection or lines of contraflexure are those lines drawn on the slab at which successive panel-wide sections of the slab cut out by vertical planes parallel to the sides of the panel change curvature from convex to concave, or *vice versa*. It is evident that since there are two such sets of panel-wide sections at right angles to each other, there must be two sets of lines of inflection crossing each other. There would also be other lines of inflection for any other sets of sections, as, for example, sections parallel to the diagonals, etc. Considering, however, for the present, only those lines of inflection arising from panel-wide sections parallel to the sides, and in order to arrive at a more complete understanding of them and their relations to the deformations of the panels, let us take, for the sake of simplicity, the case of a homogeneous uniform flat plate, such, for example, as a steel plate of indefinite extent, resting on rows of equidistant separated supports in square array, and let it be subjected to equal concentrated

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If, now, the loading is supposed to be changed from concentrated loads to a uniformly distributed load, the curvatures in the central part of the panel will not be so sharp as in the case of concentrated loads, and the lines of inflection will thereby be removed slightly toward the supports, but will still remain straight. If, however, the plate is replaced by a slab which is reinforced so as to be stiffer over the supports than elsewhere, that will move those parts of the lines of inflection which are around the supports farther away from the supports, so that they no longer will be straight. If, in addition, the stiffness of the slab across the saddle-shaped areas is made less than elsewhere, those parts of the lines of inflection which lie around the panel centers will move nearer to the sides of the panel. In such a slab, the lines of inflection will divide up the area of the slab irregularly, in checker-board fashion, into areas which are only roughly rectangular, in which the convex and concave areas have bulging sides, but the saddle-shaped areas will have hollowing sides. The sides of these irregular areas are the *loci* of no bending across them along sections perpendicular to the sides of the panel, and, consequently, of no bending moment in that direction, but that does not necessarily signify that the stresses in either the steel or the concrete is zero across these lines. This is a fact, because the steel rods which are known to be in tension both in the tension areas around the supports and in those around the panel centers, are, no doubt, in tension throughout their entire lengths, not only in the tension areas, but across the lines of inflection as well, which fact would require the entire cross-section of the concrete to be in compression across lines of inflection, because, at any vertical section subjected to bending moment only, the sum of the tensions and compressions must be numerically equal.

From what has been already stated, it is evident that the location of the lines of inflection is fixed by the rigidity of the slab, and is

largely within the control of the designer, so that it will be practically where the steel dips down below the neutral surface of the slab as it passes from the tensile zones at the top to those at the bottom of the slab. Mr. Eddy.

Now, compare this with a design in which all the steel does not dip down at the same line. The effect of this would be to cause the lines of inflection to lie at one place for one concentration of loading and at another place for a different concentration.

The difference of action between these arrangements of steel is like that existing between the cantilever bridge and the continuous bridge of several spans. In the continuous bridge, the inflection point moves along the bridge to some extent as a train crosses, whereas, in the cantilever bridge, the point of zero moments is located definitely by a joint at the junction of the cantilever and suspended span. The same advantages are secured in flat slabs as in bridges by fixing the length of the cantilever as well as the relative size of cantilever head and suspended span by having all the steel dip down at a given line, for this arrangement reduces the bending resistance at the line so as to make it the line of inflection at all times.

It is evidently possible to make the cantilever extend out from the supports to any distance, even as far as mid-span, by sufficiently reinforcing the top of the cantilever throughout and, at the same time, reducing the resisting moment of inertia of the section at the edge of the cantilever. Now, this will be practically accomplished by making all rods dip together at the same line of inflection predetermined in such position as to separate the total applied bending moment of the panel into such parts as may be desired. This may be formally stated in the following:

> Proposition III.—In a continuous flat slab on separated supports the relative size of the cantilevers over the supports and the suspended spans within the panels, as well as the relative magnitudes of the total applied bending moments across the panel-wide sections at mid-span and at the sides of the panel, may be controlled by the location of the dip in the reinforcing rods across the neutral surface at predetermined lines of inflection.

Now, it is recommended in the report to have the steel dip down one part of it at one distance from the edge of the panel and another part at another distance, thus making the position of the lines of inflection absolutely impossible of determination for any loading, and different for different dispositions of load, and also making the subdivision of the total moment between mid-span and margin uncertain. The actual subdivision will then depend on several factors besides the positions where the steel dips down, such as the relative moments

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of inertia of the cross-sections at mid-span and margin, and the stiffness of columns, etc. The recommendation is bad in principle and more to be honored in the breach than in the observance, because making the rods dip down at different points renders bending moments indeterminate which, otherwise, would be determinate. This is discussed somewhat more in detail in the next section.

6.—Column Rigidities and Relative Slab Rigidities across Panel-wide Sections, as Affecting the Sub-division between Mid-Span and Margins of the Total Applied Bending Moment in the Panel.—In any beam of constant moment of inertia, fixed horizontally at the ends and uniformly loaded with a total load, W, the applied bending moment at mid-span is $\frac{W\,L}{24}$ and at each end is $-\frac{W\,L}{12}$. In case the moment of inertia or resistance to bending is made greater at and near the ends than elsewhere, the numerical value of the applied bending moment at each end will be increased at the expense of that at mid-span, but their numerical sum will still be $\frac{W\,L}{8}$. If, however, the restraint at the ends is reduced so as to be insufficient to fix them horizontally at the supports, the positive moment at mid-span will increase numerically, while the numerical value of the negative moment at each end will be decreased by the same amount. Similar principles apply to the moments at panel-wide sections of inside panels of a continuous slab over separated supports, except that the uniformly distributed load in the case of the slab, which causes the applied moment in each direction, will be only one-half of the total panel load, W, as previously shown. The column supports of a continuous slab are not usually sufficiently rigid to afford perfectly horizontal restraint at the supports, but there seem to be clear indications that, under ordinary circumstances, the columns resist about one-half the unequalized bending moments at the supports. That, however, is partly dependent on the rigidity of the connections between the slab and the supports.

In order to obtain an outside estimate of the divergence between the effect of the complete restraint at the ends previously considered and free tipping over the supports, let us suppose a uniform beam of three equal spans resting on four supports, with a uniform load on the middle span only, but free to tip at all four supports. The theorem of three moments shows that, in this case, the applied moments at the support would be $-\frac{W\,L}{20}$, while that at mid-span would be about $\frac{W\,L}{13}$. If, however, the beam were fixed horizontally at its two ends, but could tip freely over the two intermediate supports, the amounts

over these would each be $-\frac{W\,L}{18}$, while that at mid-span would be about $\frac{W\,L}{14.4}$. These results, compared with those for the single span with fixed ends, show that, although lack of fixity at the ends of the span, L, might reduce the numerical value of the applied moment by one-third or more at the edges of the panel, at the same time it might nearly double the moment, $\frac{W\,L}{24}$, at mid-span, even when no account is taken of any additional effect due to increased stiffness, such as is usual at the edges of the panel. Mr. Eddy.

As previously shown, the foregoing results for beams should be divided by two, in order to make them applicable to inside panels of continuous slabs of equal moments of resistance at mid-span and margin.

How little the results thus obtained can be reconciled with the recommendation in the report which proposes to use the coefficient $\frac{1}{15}$, at the margin and $\frac{1}{25}$ at mid-span, regardless of relative column and slab rigidities, is left to the meditation of the reader.

7.—*The Distribution of Bending Moments Across Panel-wide Sections.*—As already stated, in connection with the bending of the crown-sheet of a boiler, the distribution of the rigidities in any panel-wide section completely determines and controls that of the intensities of the resisting moments in that section. Bending moments are by-products and secondary phenomena in a beam or slab, and depend mathematically on the shear, which is, in fact, the primary action produced by the load. The rigidities which affect the distribution of shears are mostly independent of those which affect the distribution of the moments. The rigidities which control the distribution of the bending moments do this in two distinct ways, one is by the relative resistance across different parts of any panel-wide section, which determines the distribution and relative intensities of the stresses at the several parts of that section, the other is by the relative total resistances across the parallel sections at mid-span and margins of the panel, which determines the relative total moments across those sections. The effect of the latter has been considered previously.

We are concerned here, therefore, with the sub-division of the total bending moment at one margin of a panel into two parts, one part across the two middle quarters, or inner section of the edge as it is called, and the other across the rest of the margin, or the column-head sections. The only possible basis for any sub-division or partition of the total marginal moment between these sections of the margin must be some existing or assumed distribution of the resistances across

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these sections. For, as already explained, the distribution of stresses will depend on the distribution of resistances. It should be noticed that, in case a distribution of stresses was found for a uniform plate by calculation, or by experiment, as was done by Trelease,* and then the slab was reinforced by belts of steel the resistances of which are made proportional to the stresses so found, the slab thus reinforced would no longer be one the rigidities of which were those of the original uniform plate, and the stresses in the slab would then be distributed in a manner very different from those in a uniform flat plate. So that any such basis for design is wholly illusory. This is also true regarding the subdivision of the total applied bending moment between the parts of the panel-wide section at mid-span.

The saddle-shaped deformations which occur at the sides of the panel directly between the column heads produce, in the top of the slab, tensions across the sides at mid-span and compressions parallel to the sides. These are stresses on vertical plane sections of the slab at right angles to each other, and express a state of stress in the material which may be expressed otherwise by saying that there are other vertical planes intermediate between these (inclined to them possibly at nearly 54°) on which the entire resultant stress is a shear horizontally along each plane, which increases in intensity according to its distance from mid-depth of slab.

The material of the slab effectively resists these shears, especially when it is well reinforced near mid-span of the sides, and such resistance, called into play by the load, helps to carry the load, as do all other resistances; but this is a kind of resistance additional to any occurring in any simple beam structure, and it enhances the resistance of this part of the slab above that of simple moment resistance just as truly as the circumferential and radial action of the steel does in the convex and concave areas. It is by the help of these saddle-shaped deformations that the distribution of rigidities across the sides effect a corresponding distribution of stresses, as may be seen from the following discussion.

Suppose that little or no tensile resistance existed in the top of the slab across the edge of the panel near mid-span; then the panel on each side of that edge would deflect more readily for that reason, and the deflections at the panel centers on either side of this edge would be greater than otherwise. This sag on each side and yielding to bending across mid-span of the edge would transfer the applied bending moments each way from this point and bring them to bear

* "The Design of Concrete Flat Slabs", by F. J. Trelease, *Proceedings*, National Assoc. of Cement Users, Vol. VIII, 1912, p. 218.

on the slab at and near the column heads. If, however, very rigid reinforcement is introduced across the edge at and near mid-span, it will resist the greater portion of the total bending moment across the inner section of the edge and relieve the tensile stresses across the column heads by just so much, and so prevent any necessity for piling much steel directly over the columns. At the same time it will reduce the central panel deflections as well. Until it is proved that steel across the column heads resists bending moments more economically than across the inner sections, any recommendations such as those in the report, following the Chicago ruling, and intended to control the distribution of the steel across the margin, etc., are contrary to good policy and an intolerable restraint on legitimate design. Mr. Eddy.

The truth of this last statement is evident from the following considerations. The intensities of compressive stresses across panel-wide sections must be distributed along those sections in a manner corresponding in the main with the distribution of the tensile stresses in the steel in these sections. In ordinary design, with steel belts massed across the supports, it has been frequently asserted that critical compressive stresses occur in the concrete around the column capitals. If this is true, it is due to the concentration of steel in the side belts, and can be obviated by such a disposition of the steel as to transfer a suitable fraction of the bending moments across the panel edges to the inner sections, and thereby relieve any excessive compressions in the outer sections. Any uncertainty which now exists respecting the existence of too large compressive stresses at right angles around the capitals can be thus removed, and the question of controlling the critical stresses in the concrete around the columns be thereby confined to the question of the vertical shearing stresses.

8.—Bending Moments Reduced by Size of Capitals.—In the next place, consider the distance by which the length of the span, L, between the column centers must be reduced in order to obtain a correct estimate of the effective span. How great such a reduction should be depends on several circumstances. In case the capitals are integral with the slab, and the columns perfectly rigid or nearly so, the effective span between capitals (each of diameter $= C$) would be $L - C$, and if all the reinforcing steel was to pass over these capitals, the length of the span would practically be $L - C$, while the panel load, W, would remain practically unchanged.

As a mean value, under ordinary conditions of flexibility of columns, etc., the mean effective span may be conservatively taken as $L - \frac{2}{3} C$, although this may be subject to change in special cases. This may be stated explicitly as:

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Proposition IV.—When a correction for the size of capitals is introduced, the constant applied moment in the span, $\frac{W L}{8}$, becomes approximately

$$\frac{W L \left(1 - \frac{2 C}{3 L}\right)}{8}.$$

If the panel, however, is one of a tier of panels supported on a series of successive transverse walls, each of width $= C$, instead of on separated supports, the load, W, would also need to be reduced in the same ratio as the span, and the constant applied moment per span thus corrected would become

$$\frac{1}{8} W L \left(1 - \frac{2 C}{3 L}\right)^2,$$

due to an effective span, $L \left(1 - \frac{2 C}{3 L}\right)$, and an effective panel load, $W \left(1 - \frac{2 C}{3 L}\right)$. It is evident, however, that this last result, which has been adopted in the report, is inapplicable to the case of the panels of a slab on separate supports.

Common practice makes C not less than $0.2L$, while $C = 0.225$ is a usual value, and we have, consequently

$$0.87 > \left(1 - \frac{2 C}{3 L}\right) > 0.85 \text{ and } 0.76 > \left(1 - \frac{2 C}{3 L}\right)^2 > 0.72.$$

Hence, it appears that, by reason of the size of the capitals alone, the constant applied bending moment may possibly be reduced to

$$0.85 \frac{W L}{8}$$

independently of any other reductions that may occur, such, for example, as that treated in the previous sections. However, the amount of reduction proposed in the report, which would make the total constant applied about $0.75 \frac{W L}{8}$, or about $\frac{W L}{11}$, is evidently unwarranted by theory, even in case the circumstances were such that the span were reduced by the entire diameter of the capital, for the minimum span would then be

$$L - C = L(1 - 0.225) = 0.775 L.$$

9.—Recapitulation.—First.—The formula proposed in the report for the thickness of slabs does not conform to accepted usage in reinforced concrete design, which increases the steel ratio with increase in the ratio of thickness to span. Consequently, the formula for thickness makes the proposed minimum thickness too large for large loads and spans.

Mr. Eddy.

Second.—A continuous flat slab on separated supports is an indeterminate structure in which the distribution and relative intensities of the stresses depend on the relative rigidities of the parts acting, and, in particular, on the massing and location of the steel, so that the statical limitations applicable to ordinary statically determinate structures are not valid in such a slab.

Third.—On account of the necessary sub-division of the total shear arising from the panel load which is carried in the two directions parallel to the sides independently, the total applied bending moment acting parallel to each side is only 50% of that stated in the report in case of inside panels, and 60% of it in case of outside panels, provided the latter be taken to be 20% greater than the former, as recommended in the report.

Fourth.—The recommendation that the dip of the reinforcing rods from top to bottom of slab be distributed at various distances from the edges, is not well founded. A dip common to all neighboring rods should be used to fix the position of the lines of inflection and, at the same time, to fix the sub-division of the constant total applied moment between mid-span and margins.

Fifth.—The recommendation that the cross-section of steel across panel-wide sections at mid-span and margins be apportioned in certain proportions between the inner sections and the sections at the column heads, is without basis in sound theory, and should be cancelled.

Sixth.—The coefficient of reduction of the total applied moment due to the usual diameter of capitals is approximately $\left(1 - \frac{2}{3}\frac{C}{L}\right) = 0.85$ and not $\left(1 - \frac{2}{3}\frac{C}{L}\right)^2 = 0.75$, as recommended in the report.

Mr. Turner.

C. A. P. Turner,* M. Am. Soc. C. E. (by letter).—Mr. Scott analyzes the report of the Joint Committee in respect to the modulus of elasticity of steel embedded in concrete beams on the assumption that the steel furnishes the entire tensile resistance due to the moment of the applied load. This assumption is not correct. Furthermore, the assumption that the tensile strength of the concrete furnishes that portion of the tensile resistance necessary to produce a static balance of the moments not provided for by the steel is likewise incorrect.

This latter assumption, although incorrect, is found in almost every treatise on reinforced concrete published either in America or abroad. There is one treatise, however, that by Charles F. Marsh, M. Am. Soc. C. E., which shows that the distribution of stress in a reinforced beam is not analogous, or even approximately analogous, to that of a homogeneous beam in respect to any uniform distribution of intensity of the stress transversely of the beam between

* Minneapolis, Minn.

Mr. Turner.

reinforcing bars. All agree that there is a co-action between the metal and the concrete in a beam, brought about by the resistance of bond stress, which prevents the steel from slipping in the concrete matrix. Under uniform load in a simple beam, all agree that bond stress is zero at mid-span and increases toward the end of the beam. Now, this bond force or bond stress is a horizontal shearing stress along the surface of the steel. Consequently, it sets up tensions and compressions at 45° to the surface of the metal; the intensity of these indirect stresses is greatest next to the surface of the bar, and diminishes, according to the law of distribution of force through mass, with the distance away from the surface. The deformation of the concrete due to the bond is greatest near the bar, thus forming a cone of deformation about the bar, and smaller deformations or strains occur between consecutive bars in the plane of the reinforcing steel than take place at the surfaces of the bars themselves. This condition of deformation causes some of the energy of internal work to be performed at an angle to the axis of the bar, or laterally, thus differentiating the reinforced concrete beam from the beam of homogeneous material in relation to the storage of internal energy.

It cannot be maintained consistently that burying the steel in the concrete changes its molecular properties or its modulus of elasticity, because, when we dig the steel out after it has been embedded in the concrete, we find that it is just the same kind of steel as it was before it was embedded, and possesses the same properties; and any conclusion to the contrary is unwarranted. It cannot be maintained that the joint action of the combination is equivalent to the resistance furnished by a plain concrete beam without any steel whatever plus the resistance of the tensile stress in the steel multiplied by its lever arm, because experiments in which the concrete is divided or separated at mid-span of the beam by pieces of tin so that no tensile stress can be transmitted in the tensile zone from one-half of the beam to the other by the concrete, show to a large extent the same phenomena of apparent but not true increase in the modulus of elasticity of the steel when measuring its deformation at mid-span and right across the line where it is dissociated from the concrete.

The subject of horizontal shears in vertical planes is one that is little understood by the Engineering Profession, or by theoretical writers on the subject of reinforced concrete. Lack of familiarity with this kind of action has led to an erroneous report by the Joint Committee as to the strength of reinforced flat slab and column construction, and to the unwarranted assumption in the report that the tensile strength of plain concrete in the slab can account for the enormous increase in strength which it exhibits over beam action in the column-supported flat slab. Less assumption and more experi-

Mr. Turner.

mental work and a careful discussion thereof would have resulted on the part of the Joint Committee in a far different report, and one which would not be questioned, as this one will be by all who have had any practical experience in the building of structures of this type.

This phenomenon of reinforced concrete is not so simple and apparent as it may now seem, even after the explanation has been discovered. It required nearly 3 years of hard study and investigation before this satisfactory explanation was obtained. This question was discussed in a paper presented to the Boston Society of Civil Engineers, in September, 1914, in which the phenomenon was compared in action to a thrust applied at the end of the beam. The law of conservation of energy applied to the problem would prevent a consideration of the tensile stress in concrete multiplied by its lever arm as balancing that portion of the applied moment not carried by the steel, because of the lateral action introduced in the cone-shaped deformation through the operation of bond stress about the bar toward the end. These internal shears seem to be somewhat equivalent in their action to the effect of a thrust applied at the end of a beam, and not greatly interfered with by cutting the concrete through the tensile zone at mid-span.

Although the operation of horizontal shears in vertical planes is not to be depended on in a beam, because the utility of this action is destroyed by the cracking of the concrete toward the end of the beam, the conditions are different in the continuous flat slab floor, where the tendency to rotate is predominantly about a vertical instead of a horizontal axis, and, for that reason, is dependable.

A further point of interest lies in the fact that although every pound of pull in the steel is brought on it by bond stress, the work of internal deformation in tension is largely stored within the steel, a conclusion which was far from being apparent in the first consideration of the subject. Static balance merely requires equal opposing forces, with no restriction as to the amount of work performed directly by these forces, so that although the steel may store very largely the internal work of tensile deformation, horizontal shears in vertical planes may have an equal effect in reducing deflection without performing the same relative amount of work, since they operate in a radically different manner and through a much shorter range of action.

The writer presented some tests at a meeting of the Concrete Institute relative to this subject several years ago, but has not been able to supplement them by further experimental work, as he had hoped to do by this time.

Mr. Scott deserves the thanks of the Profession for clearly bringing out the incongruities of the Joint Committee's theory which he notes

Mr. Turner.

and which the writer has endeavored to explain. Figs. 11 and 12 show, in an exaggerated manner, the condition discussed.

The impossibility of assigning any dependable value to the tensile strength of the concrete in a floor may be made apparent by the following experiment conducted by the writer. A slab, about 24 ft. square and 5½ in. thick, supported on the edges on beams and piers and by a center pier, was loaded as shown in Fig. 13. It carried this load, consisting of concrete barrels filled with water, without

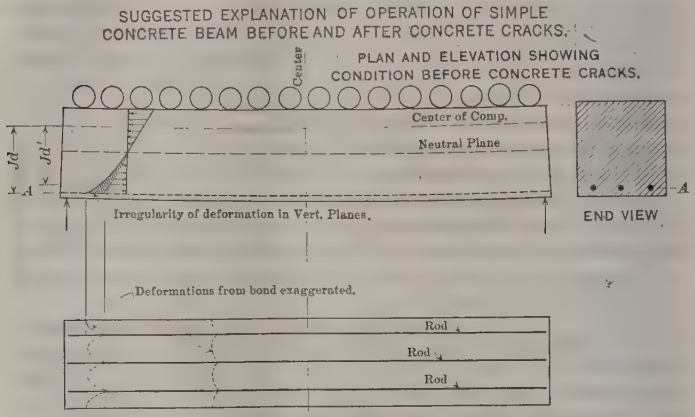


FIG. 11.

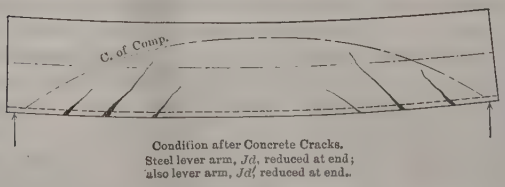
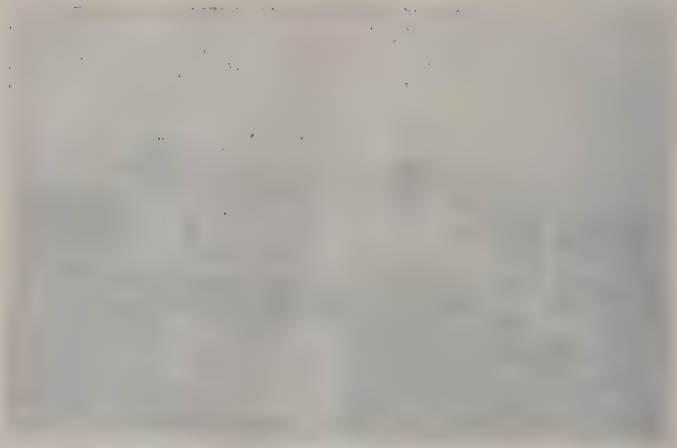


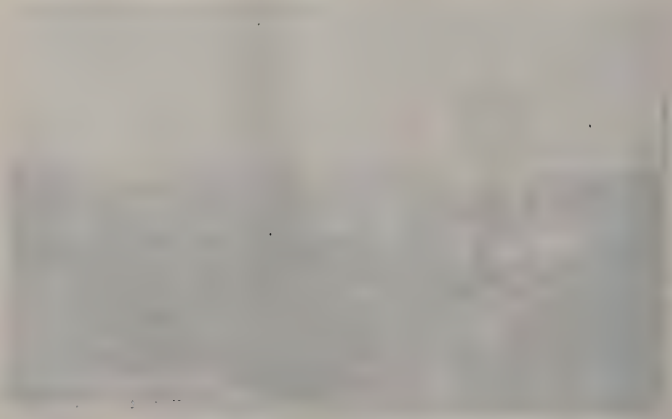
FIG. 12.

signs of failure when first applied, but a drop of temperature of about 25° caused the failure shown in Fig. 14. If a change or drop in temperature of only 25° is sufficient to overcome the tensile strength of concrete, even when assisted by a little wire netting, as was the case here, the error of relying on such resistance as furnishing any dependable load-carrying capacity in the average building, which is subjected to greater reduction of temperature than this test slab was, is apparent, and needs no further comment.

FIG. 13.—SLAB CARRYING LOAD WITHOUT DISTRESS, VERY LARGELY BY VIRTUE OF THE TENSILE STRENGTH OF THE CONCRETE.

FIG. 14.—FAILURE OF SLAB SHOWN IN FIG. 13.





Mr. Turneaure.

F. E. Turneaure,* M. Am. Soc. C. E. (by letter).—Professor Eddy's discussion of the report of the Joint Committee raises certain fundamental questions which should be answered. It appears to the writer that his analysis at certain points is absolutely incorrect, and should not go unchallenged. In the first place, on page 1178, he criticizes the Committee for applying the theory of statics to the flat slab, and for assuming flat slabs "to be statically determinate structures." The Committee makes no such assumption, and neither does Mr. Nichols. The Committee is well aware that such structures, like continuous beams, are not statically determinate, but it does not follow that the law of statics cannot be applied to them. To be statically indeterminate simply means that the unknown quantities in the problem are too numerous to be determined by the equations of statics alone. Certain total shears and moments, however, can be determined by statics, although the exact distribution of these shears and moments cannot be thus determined. This is the position of the Committee.

It is interesting to note that, after criticizing the Committee for applying the laws of statics to this case, Professor Eddy, on page 1179, proceeds to do the same thing, where, in Paragraph 3, he applies the laws of "equilibrium" to determine total shears—a perfectly correct procedure, and exactly the same sort of thing as that done by the Committee. The same process can be used to determine total bending moments along either axis, and it is quite unnecessary to make use of any involved mathematical process to do this. Although the total bending moments can be determined in this way, their distribution along the slab cannot, and, in this degree, the problem is statically indeterminate.

Professor Eddy finds that the total shear around the edges of a square slab is equal to the total load—a result with which the writer fully agrees—but the results obtained by his analysis for total bending moments are quite erroneous. For a square panel, he finds that the numerical sum of the positive and negative bending moments along the center line, and along one edge, is $\frac{1}{16} W l$, whereas the correct value is $\frac{1}{8} W l$ (neglecting the effect of size of column in reducing slightly the bending moment). As this question is a very fundamental one, and as the conditions of the problem may easily result in confusion in applying the ordinary laws of statics, it will not be out of place to discuss this matter on a rather elementary basis.

* Madison, Wis.

Mr. Turneaure.

Consider a square panel, Fig. 15, loaded uniformly with a total load, W. Assume that the columns are equivalent to square columns, in accordance with Professor Eddy's conclusions on pages 1179-80. For simplicity, consider that the columns are small as compared to the size of the panel, so that the size of the columns need not enter into the expression for bending moments. The total shear at the columns will be W, and the shear on each of the surfaces, AB, BC, etc., will be $\frac{1}{8} W$. There will be no shear on the lines, AG, CD, etc. These statements agree with Professor Eddy's conclusions.

Now proceed to the question of moments: Consider the half-panel to the left of the center line, JK, Fig. 16. There will be bending moments at all points of the periphery, these moments acting in a direction at right angles to JK along the surface, JK, and also along the surfaces, AB, CD, and EF. Moments along the surfaces, AJ, CB, DE, and FJ, will act in a direction parallel to the axis, JK. The load on the half-slab is $\frac{1}{2} W$ and the shears are $\frac{1}{8} W$ on each of the surfaces, AB, BC, etc. Now, taking moments about the axis, JK, the external moment is that of the couple formed by the load, $\frac{1}{2} W$, and the four shears of $\frac{1}{8} W$ each. This moment is practically equal to $\frac{1}{8} W l$. For equilibrium, the sum of the internal moments must also equal $\frac{1}{8} W l$, and the internal moments in question are the positive moments along JK and the negative moments along AB, CD, and EF. That is to say, the numerical sum of the negative moments along a line through the columns and the positive moment along the center is equal to $\frac{1}{8} W l$, as given by the Committee.

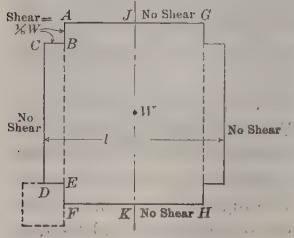


FIG. 15.

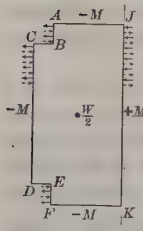


FIG. 16.

This method of analysis is a simple, direct application of the laws of equilibrium, quite as definite as the application of these laws in determining shears. No amount of mathematics will enable the conclusion to be reached that the internal moments acting perpendicular

to the surfaces, AJ, CB, etc., can help balance the bending moments about an axis, JK, at right angles thereto. Mr. Turneaure.

In contrast with this result of $\frac{1}{8} W l$, Professor Eddy arrives at $\frac{1}{16} W l$ by a process of analysis in which he omits the effect of the shears on the surfaces, CB and DE, considering that these shears affect only the bending moments parallel to JK. As a matter of fact, the shears on CB and DE are a little more effective in producing bending moments about the axis, JK, than those on AB and CF, on account of the increased lever arm. The fallacy of Professor Eddy's position may be brought out clearly by noting what happens if we consider the portion of a slab limited by the lines, AF and GH, Fig. 15. Neglecting the small load applied directly to the strip, $BCDE$, and the similar strip on the right, the total shears on the lines, AF and GH, must be equal to the load, W, by the same process of reasoning as that heretofore used. Then, if we consider one-half of this slab and take moments at right angles to the axis, JK, as before, the external moment will be plainly $\frac{1}{8} W l$, which must be equal to the total internal moments along the lines, AF and JK.

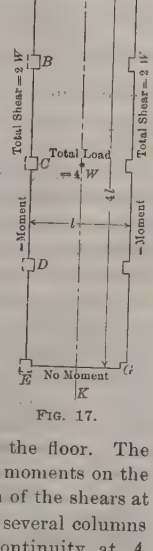


FIG. 17.

The inevitable conclusion, therefore, is that the numerical sum of the negative and positive moments along the edge and center line of a square panel is approximately $\frac{1}{8} W l$, and not $\frac{1}{16} W l$.

The matter may be clarified still further, perhaps, by considering an entire row of panels across a building. Suppose, for example, that the floor is four panels wide. In Fig. 17 is shown a row of four such panels, AF and EG being the exposed edges of the floor; AE and FG are sections through the floor. The columns are at A, B, C, D, etc. There are no bending moments on the edges, AF and EG. The total load is 4 W, and the sum of the shears at Columns A, B, C, D, and E is 2 W. The shears at the several columns are no longer exactly equal, on account of the discontinuity at A and E, but the sum is the only quantity needed. Now, take a section through JK, and consider moments on the left. The external moment is $2 W \times \frac{1}{4} l = \frac{1}{2} W l$. This is equal to the sum of the internal

Mr. Turneaure.

moments on the lines, JK and AE, acting perpendicularly to JK. The average moment per panel will be one-fourth of this, or $\frac{1}{8} W l$, as previously found. This result seems very clear, as, in this case, there are no internal moments along AF and EG to confuse the issue.

This analysis leads to the conclusion that the sum of the moments on JK and AE is precisely the same ($2Wl$), no matter what the number and spacing of the columns along AE and FG may be, as long as the arrangement is the same on each side, so that there is no shear on the line, JK. If the supports consist of continuous walls along the lines, AE and FG, the same result will be reached.

This leads to the general conclusion that, with symmetrical conditions with respect to the axis, JK, the sum of the internal moments along AE and JK is precisely the same whether the supports be walls, or columns spaced in any manner, a result which is not incompatible with the fact that the structure is statically indeterminate. Professor Eddy seems to agree that the bending moment for wall supports in the case considered would be $\frac{1}{2} W l$, but maintains, in effect, that if we take away most of the wall, leaving only small columns, the moment will be immediately reduced to $\frac{1}{4} W l$. As a result of this, one-half of the reinforcing rods may be taken out and placed in the other direction and the slab be made considerably thinner. This is an astonishing result, and obviously incorrect. The interior moments in the slab acting parallel to the axis, JK, cannot help out the moments transverse thereto. They have their own duty to perform, namely, to carry the load from column to column in the longitudinal direction, a result brought about by changing walls into columns.

Mr. Talbot.

A. N. Talbot,* M. Am. Soc. C. E. (by letter).—Professor Eddy's discussion contains so many fallacies and so many violations of accepted principles of mechanics that it is hardly believable that it is presented seriously by one who is familiar with the analysis of the action of structural parts. Since reasoning presented in mathematical form and coming from one of high standing may appear plausible, the erroneous statements and conclusions may mislead, and a more explicit answer is needed than will seem necessary to some readers.

Professor Eddy puts forward the astonishing statement that "any assumption of the validity and applicability of statical analysis to continuous flat slabs is incorrect, and leads to erroneous results, just as much as in the case of continuous beams or of any other indeterminate structures." That statical principles are valid for all structures is generally accepted, and it is difficult to believe that Professor

* Urbana, Ill.

Eddy intends to deny this generally accepted truth. Static analysis does not reveal all the conditions of indeterminate structures. For example, it cannot give the distribution of the intensity of the resisting moment along sections of the flat slab. Nevertheless, the indeterminate structure must be amenable to the statical laws of forces and moments, even though Professor Eddy seeks to exempt the flat slab from such laws. Mr. Talbot.

On page 1183 he gives a mathematical derivation of what he terms the total applied bending moment. In this derivation he uses the accepted relation between vertical shear and rate of change of bending moment, $dM = S_x dx$. In determining S_x, he adds the total vertical shear at a section, AB (Fig. 18), and the total vertical shear at a section, CD, and equates this sum to $\frac{Wx}{L}$ — the latter being the load between one of these sections and a middle section of the panel. It is evident that $\frac{Wx}{L}$ is the value of the total vertical shear on either one of the two sections, and not that of the sum of the total vertical shears on the two sections. He does not explain why he unites the shear of these two "free bodies" in the expression and yet considers the external load on only one. By this violent assumption, at one stroke the bending moment found in his analysis is cut in two. If the analysis was in any way defensible, the resulting moment coefficients would produce a wonderful saving in the dimensions and costs of structures. As the analysis involves a gross violation of the principles of mechanics, one must brand it as fallacious. That the sum of the numerical value of the positive moment at a mid-section and the numerical value of the negative moment at an edge-section is $\frac{1}{8} Wl$, as a limiting value for very small capitals, and not $\frac{1}{16} Wl$, as given on page 1184, rests on as firm a basis of logic as do any of the accepted expressions for bending moment in beams, girders, and other structures.

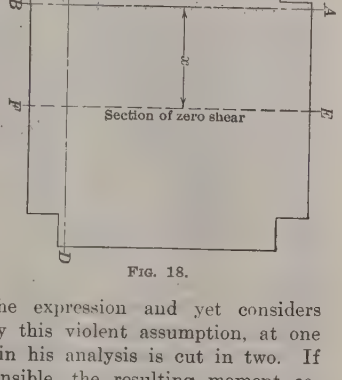


FIG. 18.

On page 1181 Professor Eddy states that the total vertical shear in sections parallel to the sides of the panels of flat slabs is 50% less than in a wide beam structure which rests on parallel walls, and

Mr. Talbot.

that the total vertical shear in a section parallel to one of the sides of a panel is only that which arises from a uniform load of $\frac{1}{2}$ W. This means that the vertical shear along a line, AB (Fig. 19), close to the supports, is only $\frac{1}{4}$ W, or less. It is plain that such a statement violates the principles of equilibrium of forces acting on the portion of the slab, $ABFE$, principles which are just as applicable with a continuous slab having sections of zero shear along boundaries, except at the supporting columns, as they would be for an isolated panel. It seems too elementary to require pointing out that, for a uniform load over the whole panel, the total vertical shear on the section, AB, is once the load between AB and CD and not one-half of that amount. The presentation made by Professor Eddy assumes that a series of vertical forces along a side of a square which lies in a direction parallel to one side of the panel (shear along one edge of a square column) does not enter into the determination of equilibrium for sections at right angles to this side. This method of cutting the shear in two is not even ingenious, and it seems hardly necessary in a discussion among engineers to add that it has no logical basis. Of course, the acceptance of Professor Eddy's value of the shear would result in dividing the moment by two.

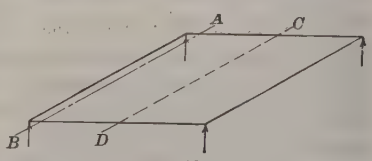


FIG. 19.

Attention may be called to the different view put forth by Professor Eddy in a paper* before the American Concrete Institute in 1916. In this paper he derives and accepts the value $\frac{1}{8}$ WL as the applied bending moment each way in a panel (the limiting value for very small capitals), instead of $\frac{1}{16}$ WL, as given in his discussion. Further on in this paper, starting with $\frac{1}{8}$ WL, he reduces the amount of moment necessarily provided for by a series of steps; (*a*) he divides it by two, because for two sets of reinforcing bars at right angles to each other, embedded in a matrix of concrete, the effect is the same as if the two sets of bars were a sheet of metal of the same weight, and hence a set of bars will have the same resisting effect laterally as longitudinally; (*b*) he divides it by two again, because as the bars do not slip in the concrete, the concrete will do as much work in a given

* "A Further Discussion of the Steel Stresses in Flat Slab Floors," *Proceedings*, Am. Concrete Inst., Vol. XII.

direction as the steel; and (*c*) he states that tests appear to show that the effect of Poisson's ratio reduces the stresses in the reinforcing bars of the slab to three-fourths of what they otherwise would be. The combined effect is to divide the moment by $5\frac{1}{3}$, and he gives $\frac{3}{128}WL$ as a final value for the sum of the positive and negative resisting moments, on which to base the design of the reinforcing bars, instead of $\frac{1}{8}WL$ before referred to. If these considerations were added to the effect of halved shear given in his discussion already referred to, the result ($\frac{3}{32}$ of that given by static analysis) would produce moment coefficients which would effect remarkable economies in structural design. Mr. Talbot.

Professor Eddy favors bending all reinforcing bars downward at a point fixed at the same distance from the panel edge. Although the position of the point of dip may be expected to affect that of the point of inflection somewhat, the differences produced for ordinary conditions will be slight, especially as the stiffness of the concrete in compression and tension has considerable influence. The recommendation in the report provides for such changes in the position of the point of inflection as may result from a change in the distribution of the load on the floor or from its concentration, and adds to the diagonal tension resistance of the slab, as well as avoids a possible plane of cleavage.

The actual distribution of the intensities of the resisting moments along a section of the panel will be affected by variations in resisting stiffness in different parts of the panel, but, as the concrete of the slab in its compressive resistance, and to a certain extent in parts of the slab in its tensile resistance, is a great factor in making up the stiffness of the slab, changes in the location or distribution of the reinforcing bars over a panel-wide section will not affect the distribution of intensities of resisting moment to any such extent as is claimed by Professor Eddy. The report allows a considerable leeway in the manner in which the reinforcing bars may be distributed over a section. It may be added that experiments have not borne out the claim for added strength in saddle-shaped portions.

Professor Eddy objects to the recommendations made for general thickness of flat slabs on the ground that it has been indubitably established that the ratio of steel reinforcement to concrete may be made greater in beams of relatively large depth over that permitted in beams of relatively small depth. The attempt, on page 1169, to establish this statement by the relation between the sharpness of the curvature and the horizontal shearing distortions in the concrete at points distant from the sections where maximum moments occur, is

Mr. Talbot.

absurd. If true, the criticism would apply also to the principles of design of all beams and girders. Besides, the relation of thickness to length existing in the ordinary flat slab is that of relatively long beams. In establishing values for the formula for slab thickness, Professor Eddy starts with two assumed examples of thickness of slab for definite panel lengths, considered to be known from experience, and, later (page 1174), he naïvely suggests that it will be sufficient to assume that l (given as the total panel length in the derivation of formulas and Table 3) is the effective span (a dimension shorter than the panel length) and may be so regarded in using Table 3. Professor Eddy seems to have overlooked the fact, which a little study will show, that the formulas given in the report for thickness of slabs with and without dropped panels are devised to produce working stresses, in both reinforcing bars and concrete, which are in close agreement with the values of working stresses ordinarily used in design when the moments and their distribution are taken as recommended in the report.

On page 1191, Professor Eddy first uses $L - C$, and then adopts $L - \frac{2}{3} C$, as the span of the panel in determining the effect of the size of capitals. The expression for moment given in the report corresponds closely to the moment of the couple formed by the resultant of the load on the half panel and the resultant of the shears at the column supports. A more exact expression is found in the paper by John R. Nichols, Assoc. M. Am. Soc. C. E., on "Statical Limitations Upon the Steel Requirement in Reinforced Concrete Flat Slab Floors."* That given in the report is a close approximation to the more exact expression.

Any information or analysis which will furnish grounds for reducing coefficients and dimensions of engineering structures should be welcomed. To be acceptable, such grounds must stand the tests of the fundamental principles of mechanics and of the known action of structures. Doubtless, the future will see many developments in concrete and reinforced concrete. With increased knowledge and experience in construction, in the choice of materials, and in the methods of work, and with the greater confidence which may develop with continued use, it is not improbable that, in the future, it will be found proper to increase allowable working stresses to more than those now recommended, and even to encroach on the margin between allowable loading and the critical load which a structure will stand—the so-called factor of safety. Practice will change, and recommendations must be revised from time to time, but changes should not be based on misapplication of basic principles.

* *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 1673.

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Paper No. 1399

FINAL REPORT OF THE SPECIAL COMMITTEE TO INVESTIGATE THE CONDITIONS OF EMPLOYMENT OF, AND COMPENSATION OF, CIVIL ENGINEERS*

WITH DISCUSSION BY MESSRS. H. S. SCHICK, LEWIS A. JONES, AND R. S. WISE.

THE AMERICAN SOCIETY OF CIVIL ENGINEERS,
220 WEST 57TH STREET, N. Y. CITY.

GENTLEMEN: The Special Committee to Investigate the Conditions of Employment of, and Compensation of, Civil Engineers submitted its second report at the Annual Meeting held on January 20th, 1915. With that report there was presented statistical information based on 4 796 replies to circulars sent out by the Committee, the information being given in diagrammatic form and covering the following:

1.—Maximum, minimum, and average yearly compensation, together with that of the middle man or the man occupying a position where equal numbers of individuals receive a greater and a less annual compensation. These figures were given for every year of experience from 1 to 63 years, although the number of replies received from those who have been in active practice for more than 47 years were so few that the information with respect to them was considered of little value.

2.—The average yearly compensation according to the nature of employment, classified in eight groups, namely, States and Counties, National Governments, Municipalities, Technical Schools, Railroads, Private Companies, Consulting Engineers, and Contractors.

3.—A geographical classification arranged in six groups, namely, Southern States, Western States, New England States, Central States, Middle Atlantic States, and Foreign Countries.

* Presented to the Annual Meeting, January 17th, 1917.

4.—A classification separating graduates of technical schools from non-graduates, this being supplemented by a statement giving the relative ages of graduates and non-graduates for those whose experience covered from 2 to 15 years.

The report of the Committee was received and the Committee was continued. The Committee has endeavored to secure information concerning a larger number of engineers, and, with this in view, has extended its inquiries outside the membership of the Society. While additional requests for information were sent out and many replies were received, the Committee was unable to present a formal report at the Annual Meeting of 1916, and simply reported progress.

In accordance with the expressed wish of the Board of Direction, that the work of Special Committees be completed as soon as possible, the Committee now presents its final report.

During the summer of 1915 the Committee addressed the same questions which were addressed to those whose replies formed the basis of its last report to those who had become members of the Society since the last preceding circular was sent out, the number of inquiries so sent being 599, from which 263 replies were received. Inquiries were also addressed to 6 393 non-members of the Society, whose names were secured from the membership lists of fifteen local engineering organizations in different parts of the country, after having first eliminated from these lists all who were members of this Society. From these non-members 1 319 usable replies were received, which, together with the additional members of the Society heard from, brings the total number of returns received by the Committee to 6 378.

The Committee has considered the advisability of addressing letters of inquiry to members of the other National Engineering Societies, but has concluded that this would be inexpedient, for the reason that there are quite a number of members of these societies who are also members of the American Society of Civil Engineers, and they are probably those whose compensation or income from professional work is above the average, so that, if those who are members of this Society were excluded, the result would not indicate the average compensation of members of the other societies, and might be misleading. The Committee also made an effort to secure information from the different engineering schools, but it was found that, with two exceptions, the statistics collected by these institutions concerning their graduates, with respect to the matters which have been the subject of the Committee's investigations, were so incomplete that this idea had to be abandoned. An attempt was made to secure pertinent data from officers of railway and other corporations, municipalities, States, and Federal departments which employ large numbers of engineers, but

they appeared to consider it impracticable to undertake the collection of such data for the Committee, and, without their active co-operation, it has not been possible to secure any information of value.

The Committee, therefore, is obliged to base its third report on the information received from the 6 378 engineers who have replied; and that these figures are worthy of serious consideration is obvious from the fact that the 5 059 members of the Society represent a gross annual professional income of not less than $20 952 952, while the 1 319 non-members represent a gross annual income of $4 467 709, or a total for those whose replies form the basis of our report of $25 420 661.

The returns from the 263 additional members of the Society, the great majority of whom were younger men, when combined with those covered by the last report, result, as might have been expected, in lowering the average compensation from $4 224 to $4 142, or $82. The average annual compensation of the 1 319 non-members is $3 387, or $755 less than for the members of the Society, including the latest returns. The Committee realizes that in directing its inquiries to individuals outside of the membership of the Society, but members of local engineering organizations, it has addressed a number of men who are not really practising engineers, as the local engineering organizations almost always include in their membership men who may be simply interested in engineering and some who have done little, if any, professional work, and they have been frank enough to say so in their returns, and their replies are not included in our statistics.

Twenty-two of the replies indicated that the men who made them are employed as sales engineers or in other lines of commercial work, although they are graduates of engineering schools. These men appear to have turned their attention to this kind of work immediately after graduation. The Committee has not included these returns in its compilation, but they are so significant as to be deemed worthy of comment. They include men whose active work covers periods of from 1 to 22 years, although in all but four instances their experience in work of this character has been 10 years or less. A comparison of their earning power with that of men engaged in what is generally considered to be strictly engineering work indicates that it agrees very closely with the averages shown by the diagrams, even though the responsibilities which must be assumed are less than those of the practising engineer.

The Committee submits herewith five diagrams as follows:

1.—A diagram similar to that reproduced as Plate XL* in the last report, showing maximum and minimum average annual compensation for professional work of 6 378 engineers, including 1 319 who are not members of the Society.

* *Proceedings*, Am. Soc. C. E., for December, 1914.

2.—A diagram showing the average yearly compensation of 6 358 engineers, divided into members of the Society and non-members, together with the combined average.

3.—A diagram showing the average compensation of graduates and non-graduates, arranged according to 5-year periods.

4.—A diagram showing the average compensation of engineers grouped according to nature of service.

5.—A diagram showing the average compensation of engineers arranged in six geographical groups.

The first of the above diagrams includes all the returns which have been received. The second and third include all returns covering professional activity of 50 years or less, while the other two cover all those which are capable of being classified with certainty and which include 40 years or less of professional work.

Your Committee has considered the question as to whether exceptionally large professional incomes, say those above $25 000 a year, might not be due to some fortuitous circumstance or condition entirely apart from professional ability, and whether they should not be excluded from consideration in the preparation of the diagrams by arbitrarily reducing them to $25 000. This was tried and, while the effect was to modify very materially the curve showing maximum professional incomes, its effect on the curve of average compensation was very slight. The Committee believes that the data collected include practically all the very large professional incomes, and that an increase in the number of returns would tend to eliminate the irregularities of the curve. The Committee's conclusion has been that the returns should be shown as they were received.

A comparison of the diagrams with those presented with the last report will show that the broader scope of the Committee's inquiry has not produced any substantial changes in the results heretofore submitted. The line indicating the average compensation of non-graduate engineers falls below that showing the compensation of graduates, except during the first part of the first 5-year period, where the professional earnings of non-graduates appear to be slightly greater than those of graduates. The Committee has communicated with bodies which are collecting data concerning professional education, in the hope that it might secure information relative to the compensation of men engaged in other kinds of professional work. Its inquiries have not produced any definite results, and, so far as the Committee can learn, no data as complete as are herewith presented have been secured with respect to them. From such meager information as was obtainable, the Committee is convinced that the compensation for engineering work compares favorably with that received by men of any other profession. Engineers, like other professional

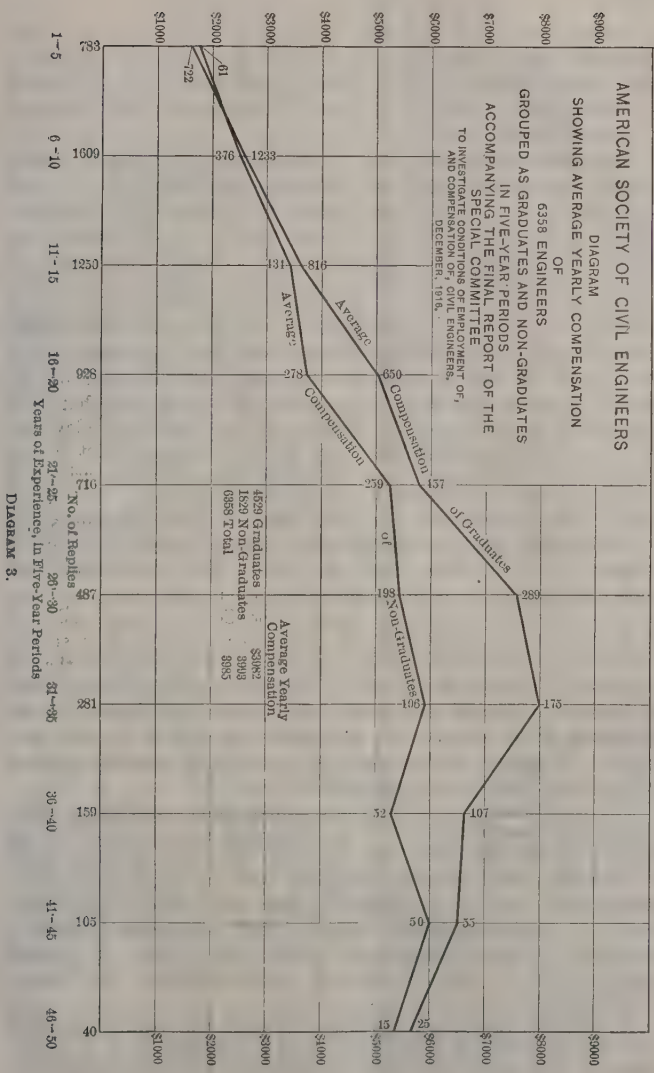

AMERICAN SOCIETY OF CIVIL ENGINEERS
DIAGRAM
SHOWING AVERAGE YEARLY COMPENSATION
OF
6358 ENGINEERS
GROUPED AS GRADUATES AND NON-GRADUATES
IN FIVE-YEAR PERIODS
ACCOMPANYING THE FINAL REPORT OF THE
SPECIAL COMMITTEE
TO INVESTIGATE CONDITIONS OF EMPLOYMENT OF,
AND COMPENSATION OF, CIVIL ENGINEERS,
DECEMBER, 1916.
Average Compensation of Graduates
Average Compensation of Non-Graduates
Average Yearly Compensation
4529 Graduates $3082
1829 Non-Graduates 3903
6358 Total 3985
No. of Replies
Years of Experience, in Five-Year Periods
1-5
6-10
11-15
16-20
21-25
26-30
31-35
36-40
41-45
46-50
783
1609
1250
928
716
487
281
159
105
40
$1000
$2000
$3000
$4000
$5000
$6000
$7000
$8000
$9000
722
61
376
1233
131
816
278
650
259
457
198
289
106
175
52
107
50
55
15
25

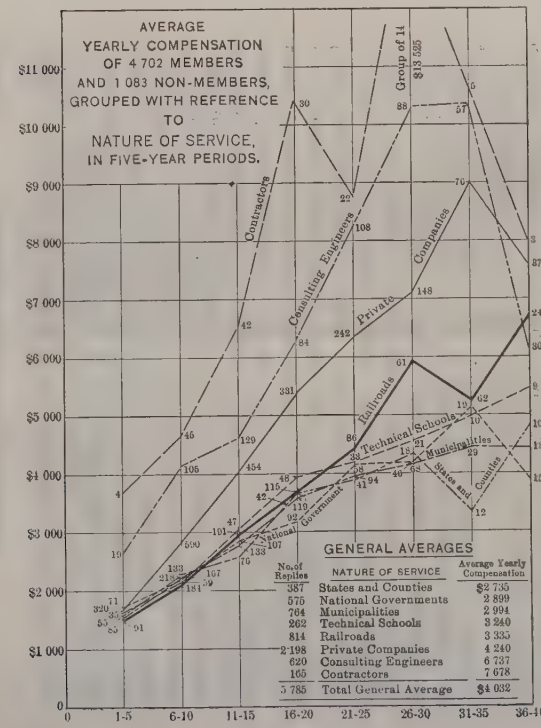
DIAGRAM 3.

or business men, are frequently desirous of being connected with important undertakings which will give them valuable experience and a certain prestige, and, in order to gain these, they are willing to accept salaries which are very small in comparison with the value of the services rendered.

The underpaid engineer does not always owe his failure to receive a salary commensurate with the importance or difficulty of his work to the ignorance or lack of appreciation of laymen, but it not infrequently is the fault of his engineering superior in the organization of which he is a part. The replies received by the Committee indicate that engineers in private practice sometimes employ men of extensive experience, and presumably of good ability, at salaries which young graduates with little or no experience are able to command, but which are less than those of an ordinary mechanic who has a labor organization behind him. It may be urged that the competition for work on the part of engineers who employ a technical staff is so keen that it is necessary to take advantage of the needs of those seeking employment in order to secure professional work which is frequently let to the lowest bidder. Engineers in private practice are often criticized because they have not established a scale of charges on a percentage basis as have the architects, but in the offices of leading architects will be found young men who are willing to work for almost nothing for the sake of the experience they hope to gain. The fact remains that employing engineers are sometimes disposed to pay their men less than their services are actually worth. The Code of Ethics, adopted by the Society by letter-ballot on September 2d, 1914, lays special emphasis upon the relation of the engineer to his client and to that of competing engineers, who may be striving to secure the same contract or commission, toward each other, but it makes no mention of the obligations of the engineer as an employer to men of his own profession. It appears to lose sight of the need of guarding against underpaying as well as against underbidding.

The engineer, in some branches, unlike other professional men, is frequently unable to locate in one place until he can become known, even though his reputation may be local. He must go where work is in progress, and, when the particular work on which he has been engaged shall have been completed, he must move on to another place. Periods of unemployment are fatal to steady advancement, while a reasonable prospect of promotion, however slow, is likely to prompt men to accept less than the average compensation for the kind of service rendered.

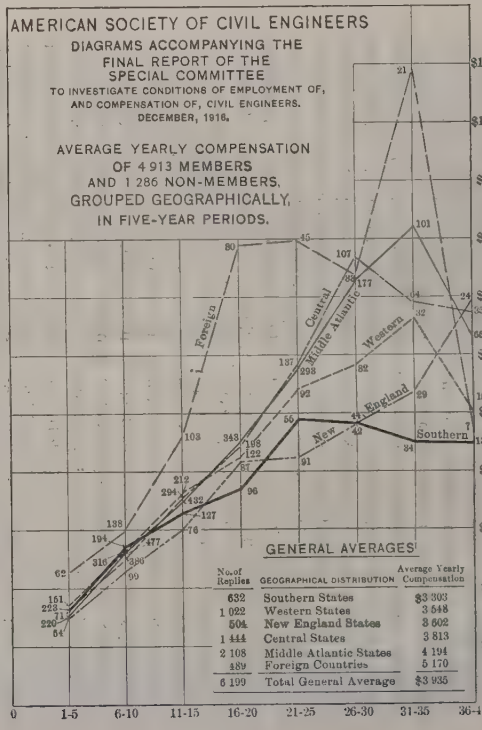
In reading the observations made by those who have commented upon the compensation of engineers and the conditions of their employment, it has appeared to your Committee that many of those which



GENERAL AVERAGES

No. of Replies	NATURE OF SERVICE	Average Yearly Compensation
387	States and Counties	$2 735
575	National Governments	2 899
764	Municipalities	2 994
262	Technical Schools	3 240
814	Railroads	3 335
2 198	Private Companies	4 240
620	Consulting Engineers	6 737
165	Contractors	7 678
5 785	Total General Average	$4 032

DIAGRAM 4.



GENERAL AVERAGES

No. of Replies	GEOGRAPHICAL DISTRIBUTION	Average Yearly Compensation
632	Southern States	$3 303
1 022	Western States	3 548
504	New England States	3 602
1 444	Central States	3 813
2 108	Middle Atlantic States	4 194
489	Foreign Countries	5 170
6 199	Total General Average	$3 935

DIAGRAM 5.

appear cynical or pessimistic have been made by men who would probably have been failures in any occupation or profession, men who do not possess adaptability or even a fair degree of industry, and who attribute their failure to their unfortunate selection of a profession. These comments should not be given much weight. Perhaps there are too many engineers, but the eagerness of young men to adopt this profession is doubtless due to the indisputable fact that young graduates of engineering schools reach a self-sustaining basis, where they are at least able to support themselves, more quickly than do those of other professions. Much has been said lately about the general decrease in the registration at engineering schools, and this has apparently been viewed with alarm by their faculties. Your Committee does not consider it a bad omen for the profession. There is a need of better trained engineers rather than of more engineers, a need more particularly of men who are better grounded in the fundamentals of the engineering sciences and who have at the same time acquired some knowledge of the economic or business aspects of engineering work, rather than of men who may have the greatest earning capacity on the day of their graduation.

The subject assigned to this Committee for investigation includes not only the compensation, but "the conditions of employment" of engineers. The information collected and herewith presented may be thought to relate only to compensation and not to conditions of employment, except to the extent that the latter may be the cause of, or may result from, the former. Judging from the replies received by the Committee and the extended comments accompanying some of them, conditions of employment are believed to be capable of expression chiefly, if not wholly, in terms of compensation. The Committee has repeatedly discussed the meaning of "conditions of employment" and what was the probable intent of the Society in the inclusion of these words in the title of the Committee. To give it the broadest interpretation to which it is susceptible and to follow the lead indicated by such interpretation would have led the Committee into a more or less controversial discussion of social conditions, and might have resulted in a report of greater length, but of no more value. That there lately has been a more general appreciation of the importance and dignity of the Engineering Profession cannot be denied, and the part which engineers are now taking, on the invitation of the Federal Government, in the investigation of the resources of this country and their prompt and effective utilization for national defense or in any other crisis, together with the more cordial co-operation between men in different branches of the Engineering Profession, will do much to increase further this estimate of the value of engineering service.

The Committee wishes to express its grateful appreciation of the co-operation which it has received from members of the Society and others, and for the valuable advice and assistance extended to it by the Secretary of the Society and members of his staff. It asks that this be accepted as its final report, and that it be discharged.

Respectfully submitted,

NELSON P. LEWIS,
Chairman.

COMMITTEE:

JOHN A. BENSEL,
S. L. F. DEYO,
DUGALD C. JACKSON,
WILLIAM V. JUDSON,
NELSON P. LEWIS,
C. F. LOWETH,
GEORGE W. TILLSON.

NOVEMBER 15TH, 1916.

DISCUSSION

Mr. Schick.

H. S. SCHICK,* ASSOC. M. AM. SOC. C. E. (by letter).—This Committee has performed a valuable service in a matter which is of first importance in the work of the Society. The diagrams accompanying the report are interesting, and their scope and limitations have been fully discussed by the Committee.

An important principle brought out in the report should receive further attention, for the purpose of emphasizing the helpfulness of the Society in promoting effectually the feeling of reciprocal usefulness, the spirit of co-operation between members, and, at the same time, perform a service of especially practical value to the profession.

The writer refers to the principle that the compensation of every engineer, aside from his return on physical capital invested, depends entirely on his own efficiency—both his technical and his selling efficiency. All know, on the one hand, as stated clearly by the Committee, that engineers frequently receive a lower compensation than their services should demand, and, on the other hand, that the search for the proper man to perform a given service is always attended with some difficulty and much loss of time. This difficulty may be charged to the fact that, in marketing their services, engineers are not always good salesmen, and have not the most ready means to acquire reliable disinterested information regarding the demand for their work. For a number of years, the American Society of Mechanical Engineers has maintained a bureau for the purpose of gathering and publishing, in several ways, just such data for the use of its members, at a cost which is surprisingly low, and with highly gratifying results.

This matter is brought to the attention of the members of this Society, now, for the purpose of eliciting discussion, and perhaps ultimately turning the valuable contribution of this Committee toward the accomplishment of a practical end which will be of much satisfaction to all.

Mr. Jones.

LEWIS A. JONES,† ASSOC. M. AM. SOC. C. E. (by letter).—There is one point in this report to which the writer desires to call attention. In Diagram 4, giving the average yearly compensation grouped with reference to nature of service, the curve for employees of the National Government is shown to increase in a fairly regular manner, with an average yearly compensation of $2 899. Attention is called to the fact that no distinction is made between army and navy engineer officers and civilian employees. It is a well-known fact that engineer officers in the Government service receive practically twice as much as civilian employees, counting the allowances for quarters, etc., and, as a result,

* Wheeling, W. Va.

† Montgomery, Ala.

the curve, in its present shape, does the civilian employees an injustice. Could not a separate curve be made for each? The writer believes that the results would show the engineer officers' curve following closely the curve labeled "Private Companies", and that for the civilian engineers well below the one labeled "States and Counties". Mr. Jones.

R. S. Wise,* Assoc. M. Am. Soc. C. E. (by letter).—This report is very complete and interesting, so far as it relates to the civil engineer, and the Committee deserves a great deal of credit for its work. Mr. Wise.

The writer is sure that a great many of our members would like to see a committee appointed to investigate and find ways and means of regulating the fees which the municipal engineer and surveyor should get for his services. As it is now, there is no schedule of fees on which such a man may base his charges.

On account of keen competition, he must base his charges on local rates, which in most cases have been established for many years; but, if rates were established, clients would not go shopping to see which surveyor would do his work the cheapest.

There should be fixed rates for establishing grades, cross-sections, and calculations of quantities on street work, for giving lines and grades for curbing, guttering, flagging, macadamizing, etc. Minimum rates for staking out lots, based on the size of the lot, for surveying and mapping out land, based on the location; one rate for the fire or congested district and another for the rural section.

There should be some understanding as to Court charges, not strictly for professional work, but for the surveyor's knowledge as to certain lines, maps, etc. In the district around Passaic, N. J., it has been the habit of some lawyers to employ the surveyor to locate certain properties, and later to subpœna him, often keeping him in Court two or three days, breaking up his field party, and not paying him for his time in Court.

* Passaic, N. J.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

TRANSACTIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

Paper No. 1400

FINAL REPORT OF THE SPECIAL COMMITTEE ON FLOODS AND FLOOD PREVENTION*

WITH DISCUSSION BY MESSRS. H. M. EAKIN, JOHN W. HILL, M. O. LEIGHTON, CYRUS C. BABB, KENNETH C. GRANT, B. F. GROAT, H. M. CHITTENDEN, MYRON L. FULLER, GERARD H. MATTHES, H. K. BARROWS, N. C. GROVER, E. C. LA RUE, FARLEY GANNETT, C. E. GRUNSKY, C. McD. TOWNSEND, AND MORRIS KNOWLES.

General.—With the growth of the United States, the question of flood control is daily assuming greater importance, but as yet has not received the attention which it merits. When a country is a wilderness, a flood occasions little damage, and is rarely recorded, but as the valleys are cleared and the farmer attempts to utilize them, the injurious effects of floods begin to be felt. With the growth of cities and villages, there arises a still greater demand for protection against overflow. While there is no conclusive evidence that great floods have increased in frequency or volume in recent years, with the growth and development of the country, the damage which is caused has been greatly intensified, and the obstruction of the flood plain by the works of man has in many cases largely increased flood heights for the same volume of discharge.

It is only in recent years that the property losses during floods have been reported, but these are rapidly increasing and becoming a serious burden on the resources of the country. Nor are they confined to any particular section. Floods of great intensity are liable to occur in semi-arid regions where the mean annual rainfall is less than 20 in., though possibly with less frequency than in regions having an average rainfall of over 60 in.; and a mountain stream with a slope of 6 ft. to the mile, is as liable to overflow its banks as

* Presented to the Annual Meeting, January 19th, 1916.

an alluvial river with a slope of 6 in. per mile. In fact, the greatest loss of life from floods usually occurs on the minor streams, whose waters rise with great rapidity and flow with great velocity.

Data Deficient.—On most streams the physical data collected have been for other purposes than flood control, and are therefore deficient in the specific information necessary for such an investigation. In a river which is being improved for navigation, the essential element requiring consideration is the low-water flow.

When a stream is to be utilized for water-power, again the first consideration is the low-water flow. If it is proposed to increase the low-water discharge by means of reservoirs, the engineer is interested in average conditions, and if he discusses those of extreme floods, it is only to provide a sufficient capacity for flood discharge without danger. Similarly, in determining the water-supply for a city, it is the low and mean flow of the stream which receives special consideration.

In discussing flood control, low and average conditions are of little value. A great flood is caused by an exceptional combination of most adverse conditions. A mean rain falling on a soil having a mean percolation and mean run-off, with a mean evaporation and other average conditions, produces about a mid-stage flow in a river. The extreme flood stage occurs when a maximum precipitation falls on a soil which because of frost or previous saturation has a minimum percolation and a maximum run-off, with a minimum evaporation. Such combinations are comparatively rare, and unless some one is specially delegated to investigate them, generally escape observation.

It is not only the causes of extreme flood discharge, however, on which additional data and further study are required. There is a large field for investigation in the influence of the works of man on the regimen of streams, and in the evaluation of all the little-understood factors that determine the flood height obtained from any given discharge.

Suggested Methods.—The methods of flood control which have been suggested may be summarized as follows:

(*a*) Methods of reducing extreme variations in rainfall and increasing the soil absorption of water, so as to diminish the run-off; reforestation, soil drainage, variations in methods of plowing, the construction of small dikes in agricultural areas, which will retain on the soil a large portion of the rainfall until there is time to absorb it.

(*b*) Reservoir and detention basins which will retain the crests of the run-off and discharge them at lower stages.

(*c*) Barriers thrown across mountain streams to prevent erosion and the transportation of débris.

(*d*) Channel improvement, cut-offs, and the construction of auxiliary channels.
(*e*) Levees which will confine the flood waters to the channel.
(*f*) Outlets for the purpose of facilitating the discharge of the streams.

Present Knowledge.—There is nothing to indicate that man can modify either the intensity or distribution of precipitation. To utilize the forces of Nature, which will reduce the effects of excessive precipitation or increase the absorptive power of soils, necessitates an intimate knowledge of the intensity and distribution of the rainfall over the drainage area to be considered, of the character of its vegetation, of the permeability and depth of the overlying soil, and of the capacity of subterranean fissures and conduits to carry off the water which may penetrate to the underlying formations—subjects on which but a limited amount of information is at present available to the engineer.

The Weather Bureau has established stations for observing the precipitation, and the results are published in its monthly bulletins. Most of these observations, however, extend over a relatively short period of time, and it is improbable that the maximum precipitation that may occur at a given locality has yet been recorded. Moreover, they are generally taken in cities on high buildings where the gauges are not protected from wind, or in villages located in the valleys, and the relation of the precipitation over a built-up territory to that of the surrounding country and upon the uplands and divides has not been well determined.

The series of studies made by the United States Geological Survey on the surface water supply of the United States are necessarily based on comparatively few observations frequently taken at less than flood stages. With such an extensive field to cover, the observers cannot wait for extreme conditions on each stream, but must pass from stream to stream measuring the discharge which happens to exist when the observer is present. In 1914 the Geological Survey made stream-flow measurements at 3 400 points and maintained 1 480 gauges in conjunction with co-operating parties; many more are needed.

Care must be exercised in the use of discharge curves or rating tables. They may represent only average conditions, and when applied to a particular case may lead to large errors. Their extension above a bank-full stage cannot be depended upon for accurate results. With sedimentary streams in particular, both the slope and the area of cross-section are subject to abrupt changes, and floating débris may land against a pier or otherwise obstruct the flow in such a manner as to entirely change the relations which exist at lower stages.

At every town subject to overflow from a neighboring stream, there is urgent need of co-operation, not only to extend the discharge

curves to extreme stages, but also to investigate the variations which may occur. On rivers which are being improved for navigation, such co-operation may be afforded by those engaged in the work of river improvement, and even though not strictly required for the development of a navigable channel, should, if necessary, receive Congressional authorization. On other streams, State commissions and private agencies are at present collecting data, all of which should be co-ordinated, studied, and published by some National agency.

But in considering flood control, the amount and character of the sediment carried is next in importance to the discharge of the stream. The advisability of reservoir and barrier construction, the practicability of enlarging the stream section, of affording relief by auxiliary channels, and of levee control, will be influenced by this factor. In streams carrying a large amount of sediment, the checking of the normal velocity by reservoir construction will cause its deposit; a sudden enlargement of the section of a stream will similarly reduce velocities and cause deposits, and this deposit is liable to occur during low stages, filling the channel on which reliance is placed for relief from flood discharge. If an auxiliary channel is built, a difference in velocity in the two channels is accompanied by a tendency of one or both to fill.

An extensive series of observations on the flow of sediment in the Mississippi and Missouri Rivers has been made by officers of the Corps of Engineers, U. S. Army, and the Mississippi River Commission, from 1879 to 1884. The Geological Survey has also published, in Water Supply Papers 236 and 274, investigations of the flow of sediment in other rivers.

Similar data on many other streams requiring flood control are lacking, and should be supplied.

A knowledge of the topography of the country, which is essential before plans for flood control can be formulated, is also lacking. In the Thirty-Fourth Annual Report of the Geological Survey it is stated that about 38.9% of the United States had been surveyed, and these surveys are generally plotted on maps of a scale of 1:62 500 and 1:125 000 (about 1 in. to 1 and 2 miles, respectively) with contours 20 and 50 ft. apart. Such surveying and mapping, even in greater detail, should be prosecuted with vigor and co-operative action by the General Government and the various States until the entire country has been covered.

For the large rivers the present maps give but few contours at wide intervals in the valleys, and the usefulness of such data in a study of flood control is therefore limited to purposes of reconnaissance. With the exception of those of the Mississippi, Missouri, and Illinois Rivers, the surveys for the improvement of navigation have been generally confined to the river bed.

The organic act creating the Mississippi River Commission stipulated among other duties that such surveys, examinations, and investigations of the Mississippi River and its tributaries should be made as might be deemed necessary for the purpose of formulating plans for the prevention of destructive floods. As a result of this legislation the Mississippi River Commission has been engaged for the last 35 years in collecting data relating to the flood control of the Mississippi River. An accurate hydrographic survey has been made of the river from its head-waters to its mouth, with a topographical survey covering practically the entire width of the valley down to the head of the alluvial basin near the mouth of the Ohio, and extending to a distance of about 1 mile upon either side, with occasional sections across the entire alluvial valley, below the Ohio. Maps have been prepared on a scale of 1:10 000 with contours 5 ft. apart. Discharge stations have been established on the main river and also on some of the larger tributaries where the flow is not affected by back-water, and the discharge curves have been developed from a long series of observations. In the earlier stages of the work, daily observations of sediment and discharge were made at several stations for periods of over a year. In later years, discharge measurements have been confined to prescribed stages above the normal high water and below the average low water, in order to trace any changes in regimen that might occur. Numerous gauge stations have also been established on the main river and its principal tributaries where the height of the river is recorded at least daily, and observations have been made to determine the slope, at both high and low water, and other hydraulic factors.

By the River and Harbor Act of June 18th, 1880, a project was adopted for the construction of forty-one reservoirs in Minnesota and Wisconsin to collect the surplus water from the precipitation of winter, spring, and early summer, and release it during low water so as to benefit navigation on the Mississippi River. In 1887 the scope of the project was reduced to include reservoirs at the head-waters of the rivers in Minnesota, only, and there have been constructed six reservoirs with a capacity of about 96 000 000 000 cu. ft. and having a drainage area of 4 535 sq. miles.

These reservoirs, while built for the low-water regulation of the river, were much larger than any others that had then been built by man for the high- or low-water control of rivers.* Careful records have been kept of the rainfall, the contents, and discharge of these reservoirs since their construction; and they afford most valuable data as to the flood control of a river by reservoirs, particularly as there is a

* Three basins on the Ottawa River now under construction will have a capacity of 168 000 000 000 cu. ft.

different relation for each reservoir between its capacity and the area of its water-shed.

The surveys and estimates for reservoirs in Wisconsin, under the original project, afford further data for the discussion of flood control of the tributaries of the Mississippi River in that State.

The California Débris Commission has carried on a series of investigations of the Sacramento River and its tributaries, which are useful in determining the flood control of that drainage area. Important studies on the Allegheny and Monongahela drainage area have been made by the Pittsburgh Flood Commission. A large amount of data on the Miami River has been accumulated, in connection with the work of the Dayton Flood Prevention Committee. The Scioto River has been made the subject of an extensive study at Columbus. And several flood commissions have been appointed by Governmental agencies, such as the Ohio River Flood Board of the U. S. Army Engineers and the Commission of the States of Indiana and Pennsylvania, though few of these have yet had sufficient opportunity to contribute any extensive studies upon stream regulation. Some of the minor valleys have been surveyed by cities to determine the proper source of a water supply, and a few corporations have determined the capacity of certain streams to develop water power; but, on the whole, there is a great dearth of reliable data as to rainfall, run-off, and other physical facts needed by the engineer for the proper solution of the problem of flood control, and the accumulation of the proper data will require many years of careful observation. Even with proper data available, the determination of the method of flood control, which is to be applied to a particular case, will require careful consideration and the greatest exercise of engineering skill. No method can be devised that will be susceptible of universal application.

Discussion of Suggested Methods.

(*a*) *Reforestation.*—If reforestation is considered merely from a commercial standpoint, the value to the country of reproducing our timber is so great that it obtains general approval, particularly of engineers who appreciate more than any other class the disadvantages to which the country will be subjected by the destruction of its forests.

But some advocates of reforestation have claimed that the destruction of our forests has increased the height and frequency of floods and diminished the discharge during low water, and have based their claim for reforestation on the beneficial influence of forests in preventing floods and improving the navigation of our rivers.

The relation of forest to stream flow has been extensively discussed in this country, in papers of recent date, by the U. S. Geological Survey; U. S. Weather Bureau; Flood Board of U. S. Engineers, on the Ohio River; Pittsburgh Flood Commission; Norton, on the Rivers

of Michigan; Chittenden, before the American Society of Civil Engineers; Moore; Burr, on Merrimac River; Mead, on Rivers of Wisconsin; and the Second Report of the National Commission for the Conservation of the Waters of the United States; and others. In Europe, in addition to the classic papers of Gustave Wex, in 1873 and 1879, Keller, Lauda, and Oppokow have recently discussed conditions in Germany, Austria, and Russia, and a paper by G. Fantoli, 1913, sums up the status in the valley of the Po. An interesting discussion of the subject will also be found in the *Proceedings* of the International Navigation Congress in Milan, 1905.

In these discussions the greatest diversity of opinion has been expressed, and even the advocates of reforestation as a means of flood control fail to give any quantitative determination of the effects of forests upon floods. For the engineer to utilize any agency practically, he must know that it will produce a positive effect on which he can rely and which he can measure. The data as to the effect of forestation, soil absorption, and kindred methods of flood control are not at present susceptible of quantitative analysis.

Over a large portion of the United States freezing occurs during winter, and a heavy precipitation may occur when the ground is covered with an impenetrable layer of ice and much of the vegetation has shed its leaves. Such conditions prevent the utilization of forestation or ground storage. This was the principal cause of the great flood of 1912 which devastated the Ohio and Mississippi Valleys.

(*b*) *Reservoirs.*—The equalizing effect of lake storage on the discharge of the St. Lawrence and Oswego Rivers is frequently quoted as illustrating efficient flood control by reservoirs. But in these cases Nature has been much more lavish in the expenditure of its resources than man can afford to be. About one-third of the water-shed of the Great Lakes is devoted to reservoir purposes.

The permanent overflowing of land which can be devoted to agriculture is costly, and reservoir construction is most economical when confined to mountainous valleys where the area of tillable land is small when compared with the volume of water that can be stored. On many streams this condition materially limits the applicability of reservoirs as a means of flood control, particularly in thickly settled districts.

It also frequently happens that a reservoir can be more profitably utilized for other purposes, such as power development, irrigation, or the water supply of a city. There is a popular delusion that the same reservoir can be utilized simultaneously to reduce floods, increase the low-water discharge of a stream, and increase the water-power that can be developed therefrom, but ordinarily its utilization for any one of these purposes precludes its efficient use for either of the others. If a reservoir is to be utilized to diminish floods, it is essential that

it be empty when a heavy rain occurs. If to increase the low-water discharge of a river, it must be full when the rainy season ceases. To insure the first condition, it is necessary to empty the reservoir after every storm, to obtain space to store the discharge of the one following. To insure the second, it is necessary to close the outlets of the reservoir at relatively low stages, and when it is once filled, not permit the surplus water to escape until the river falls to the stage when the stored water is required for navigation. Both of these processes interfere with the development of water-power, whose effectiveness is measured by the minimum discharge permitted. The cause of the trouble is the irregularity of rainfall in different years, and the necessity of being prepared for a minimum rainfall when considering navigation interests and a maximum for controlling flood conditions. In years of average rainfall, it may be practicable to utilize a reservoir for different purposes, but it is reiterated that average conditions do not enter into the problem of flood control.

Your Committee, however, does not intend to condemn *in toto* the utilization of reservoirs for more than one purpose. In fact, it believes that the practical solution of the flood problem in some valleys will be found in permitting corporations to build reservoirs in which a portion of the stored water can be utilized to a limited extent for power purposes and the remainder for flood prevention, but in such cases one of the conditions of the franchise must be the construction of reservoirs far in excess of the dimensions required for the development of the proposed power, and a rigid supervision must be exercised by the authority granting the franchise, not only to see that its requirements are fulfilled in construction, but also in operation.

There are in some localities opportunities for arresting the flow of a stream during the progress of a flood by temporary storage which limits the river discharge to that which may be safely confined within banks. This can be done by detention basins which differ from storage reservoirs in that they do not require the abandonment of the sites occupied for storage purposes. Flood control in such cases is only necessary during exceptional rains, and the basin itself may therefore be devoted to its normal uses most of the time, and only an easement on the lands occupied is necessary.

By this means cities can be protected at a minimum inconvenience to the rural community. The use of detention basins has been adopted by the Miami Conservancy District for the purpose of controlling the floods of the Miami Basin.

(*c*) *Barriers.*—Barriers erected across the valleys of tributary streams have been utilized to retain detritus and prevent its deposition in the main river. When conditions are favorable, this is a practical method of control.

(*d*) *Channel Enlargement and Cut-Offs.*—A stream flowing in an alluvial valley adjusts its channel to its discharge; when the discharge increases, the area of its cross-section is increased, and when the discharge diminishes the area of cross-section is reduced. In a river like the Missouri or the Colorado, carrying a large amount of sediment, these changes are rapid.

Where a channel is enlarged for the purpose of carrying extreme floods, it should be so designed as to give a velocity at all stages, which will prevent deposits. If not so designed, the periodic removal of deposits by mechanical means will be necessary.

A cut-off or straightening of an alluvial channel affects its slope which the stream attempts to regain, producing excessive caving of banks. Both stream-enlargement and cut-offs, while reducing flood heights immediately above them, increase the discharge capacity locally and increase flood heights below them. In alluvial rivers such work may seriously affect the regimen of the low-water channel, producing shoals, both above and below them, and should not be attempted when questions of navigation are involved or bank erosion may seriously affect other interests.

In streams of stable banks, and beds carrying but little sediment, such straightening may not be injurious.

(*e*) *Levees.*—Every alluvial stream flowing in a valley which has been formed by its own deposits, builds up its banks to a certain height which is generally known as the bank-full stage. Below that stage the stream flows in a more or less clearly defined trough and, at higher stages, it spreads over the alluvial valley. When an unleveed river rises above the bank-full stage there is an abrupt change in its regimen. The area of its cross-section increases very rapidly and ceasing to follow the sinuosities which its low-water channel has created, its currents tend to flow across points, materially changing the slope. These changes in regimen affect the movement of sediment, causing deposits at some localities and scour at others.

When the river is leveed this abrupt change in regimen is prevented to a greater or less extent, dependent on the characteristics of the river and the distance of the levee from the river bank. The function of a levee, as far as river hydraulics are concerned, is to extend the natural banks up to extreme flood stages and thus continue or possibly intensify the conditions which exist at a bank-full stage, *i. e.*, wherever a stream tends to scour or fill at a bank-full stage, that tendency may be somewhat intensified at extreme flood stages if the stream be controlled by levees which have been properly located with reference to its banks.

There is another feature in connection with levees which it is believed has been generally overlooked, and that is the increase

of channel storage which they create, and the resultant reduction of flood height from this cause. Under present conditions of levee alignment, between the mouths of the Ohio and Red Rivers, the Mississippi River has a storage capacity of 1 365 000 000 000 cu. ft. (about 30 000 000 acre-ft.).* When we consider this enormous reservoir effect, it is evident that flood control on the Mississippi River below the Ohio cannot be realized without the use of adequate levees.

(*f*) *Outlets.*—Outlets are at best of doubtful utility in reducing the flood heights of sedimentary streams, even where additional lines of flow to the sea or lake into which the stream empties are practicable. The frictional resistance of the river bed limits their influence to a comparatively short distance above them. In a delta formation they usually require leveeing to protect the adjacent country from overflow and in a river carrying a high percentage of sediment, either the outlet or the main stream will diminish its area of cross-section by fill and thus reduce the discharge capacity and ultimately defeat the purpose for which the outlet is constructed.

Three natural outlets from the Mississippi River to the Gulf of Mexico may be cited as illustrating this principle. The Atchafalaya River leaves the Mississippi at a point about 200 miles above New Orleans and reaches tide-water at a distance of 100 miles from its source, while the main river traverses a distance of 300 miles before reaching the Gulf. Although having three times the slope of the main river, the entrance to this outlet practically closed itself at low stages by natural processes of deposit and, in order to maintain navigable depths, annual dredging has become necessary, and if this were suspended the closure would doubtless in time become effective.

The Jump, an outlet which leaves the right bank of the river about 10 miles above the Head of the Passes, was once a formidable outlet, but is now of insignificant proportions; Cubits Gap on the left bank, about 5 miles above the Head of the Passes, which was cut through to the Gulf in 1862, at first enlarged very rapidly to a width of over ½ mile with a maximum depth of 80 ft. near the entrance, but has now become a shallow outlet which is being rapidly obliterated by deposits.

So far as the Mississippi River is concerned, whose regimen has been closely studied for over half a century, the observed effects of the natural outlets that have been described and the effects of crevasses in levees carrying one-fourth of the flood volume of the river with but a small reduction in gauge heights, all point to the conclusion that the use of outlets in any form for the purpose of reducing flood heights will generally fail to secure the desired results.

* The Roosevelt Dam in Arizona stores about 1 500 000 acre-ft.

Conclusions.

Your Committee therefore submits the following conclusions:

Reforestation.—The effects of forest growth in preventing erosion on hillsides are sufficient to justify reforestation for that purpose, but there has been no quantitative determination of its influence on stream flow which would justify its employment as a method of flood prevention.

Reservoirs and Detention Basins.—At the head-waters of streams, storage reservoirs and detention basins can be successfully employed to reduce flood heights. Which method is preferable is dependent on local conditions. Their efficiency, however, rapidly diminishes as the distance from them increases.

Cut-Offs, Channel Enlargement, By-Pass, and Outlets.—In clear-water streams, whose banks and beds are not subject to scour, these agencies may be profitably employed. In streams carrying a large amount of sediment, whose banks are readily eroded, great care should be exercised in their employment, always bearing in mind the fact that the best conditions on an alluvial stream follow its confinement to a single channel.

Levees.—As you proceed down stream the influence of reservoirs on flood prevention rapidly diminishes, and the influence of levees correspondingly increases in importance as a method of flood protection. On the lower alluvial reaches of long rivers, such as the Mississippi and Colorado, they afford the only sure means of flood control.

Your Committee believes that it can perform no greater service to the Profession than to call attention to the paucity of the data existing in reference to flood control and to the damage which may result from river regulation legislation, either by the Nation or the States, calling for definite projects which are not predicated on full and thorough investigation.

While valuable results have been obtained by National departments and bureaus and State and local commissions, with the limited means at their disposal, such investigations have generally been subordinated to other questions than flood control, and there is pressing need of intensive study directed especially to this end. Moreover, these agencies have worked on independent lines, and there has not been the necessary co-ordination of their observations. Progress would be much more rapid if each local investigation could be brought to follow some uniform method of procedure and to cover all of the points included in a standard outline, which would permit the fullest utilization of the data obtained.

Your Committee desires to urge the great importance of the early establishment and unification of systematic rainfall, run-off, and flood observations, covering the entire country in far greater detail than has yet been attempted.

While the co-operation of all agencies engaged in river regulation is desirable, the question of flood control is becoming of such vital importance to the country that it should not be subordinated to questions of navigation or power development. The disasters of 1913 in the Ohio Valley—involving a property loss of $67 000 000, and more than 361 lives—and that at Kansas City, in 1903—involving a property damage of at least $10 000 000—occurred on non-navigable streams, and on some of the navigable rivers the interests affected by floods are of far greater importance than those connected with navigation. If considered merely as an accessory to navigation, the limits of investigation of flood problems will be vastly curtailed, and their usefulness correspondingly diminished.

The members of your Committee have received reports from many sources covering investigations in connection with stream flow, and have been much gratified with the prompt, cordial responses to inquiries for data. In digesting this information it soon became apparent that reliable records of past floods were too meager and covered too brief a period to justify at this time a general report on the floods of the country.

A voluminous report might be prepared by compiling and tabulating areas of drainage basins, slopes of streams, gauge readings, oscillations in stage, and other physical data which the Committee has gathered. But this did not seem to be judicious, since only a small portion of our members are interested in such information and those who are would doubtless prefer to go to the official records for the more complete data required in their special work. Moreover, the expense and labor involved in preparing and printing such data would be disproportionate to the value of the results.

C. McD. Townsend, *Chairman*,
John A. Bensel,
T. G. Dabney,
J. B. Lippincott,
Daniel W. Mead,
J. A. Ockerson,
Arthur T. Safford,
Charles Saville,
F. L. Sellew,
C. E. Grunsky.*

*Endorsement of the Committee Report by C. E. Grunsky.

In concurring with the conclusions of the Committee on Floods and Flood Prevention, the undersigned desires to emphasize the fact that he is in favor of cut-offs and the straightening of river channels whenever conditions permit. The cut-off is not undesirable, as might be inferred from the language of the report, but desirable, and yet there

will be cases where bank protection may be too expensive to justify recourse to the use of cut-offs that might otherwise be thought advisable.

He desires, too, to add that, after the flood flow has been reduced by all practicable means, the aim should be:

a.—To make the stream carry as much of the flood flow as can be carried between levees of reasonable height.

b.—To allow water in excess of the maximum stream capacity to escape from the river under control at selected points.

c.—To convey the water which is allowed to escape from a river during flood stages, to a re-entry into the river or to a suitable place of outfall, under such control as will prevent undue inundation.

C. E. GRUNSKY.

MINORITY REPORT.

BY MORRIS KNOWLES, M. AM. SOC. C. E.

The writer regrets that at the last moment he finds himself in a position which requires that he submit a minority report. His disappointment is especially keen, as he was and is in substantial agreement with the report as originally drafted by the sub-committee. It is only because some portions of the report now presented represent so wide a departure therefrom and because of failure to secure such reconsideration of these changes by a reconvened committee meeting that he feels it incumbent upon himself to present these views for the consideration of the Society.

His objections to the report as now submitted relate to some expressions scattered through the paper, in which only a single view is presented on controversial subjects on which it must be recognized that much more information can and should be obtained. Such expressions, he believes, promote controversy rather than helpful discussion, and discourage rather than encourage further investigation. It would be far better to express—as is so ably done by our Board of Direction when submitting questions for vote—the reasons for and against various opinions. It is unwise to assume that we have at any time obtained all the knowledge possible upon a given subject. If we will approach the question from this point of view and with full confidence that the truth in the end must prevail, the opinions of the Society, when finally expressed, will deserve and receive recognition as authoritative.

For these reasons, therefore, the writer presents these amendments for consideration by the Society:

(1).—Page 1221, the third paragraph reads as follows:

> "Similar data on many other streams requiring flood control are lacking, and should be supplied."

In order that it may not appear that the kind of observations already made are entirely sufficient, which is probably not the case, and as there are several other items of data required for a complete study, the writer suggests that the following paragraph be substituted for that above quoted:

> "Extensive studies are needed regarding the physical and physiographic laws affecting stream loads, rates of transportation and deposition, and the variation of these factors with different conditions."

(2).—Page 1224, the first sentence, first paragraph, reads as follows:

> "In these discussions the greatest diversity of opinion has been expressed, and even the advocates of reforestation as a means of flood control fail to give any quantitative determination of the effects of forests upon floods."

This is one of the most controversial subjects upon which engineering and scientific minds have been recently engaged, and inductive reasoning does not at present result in conclusive proof in all cases upon either side of the dispute. The writer, therefore, suggests the following sentence as a substitute for that above quoted:

> "In these discussions the greatest diversity of opinion has been expressed and neither the advocates nor the opponents of reforestation as a means of flood control have completely established to universal satisfaction a quantitative determination of the effect of forests upon floods."

(3).—Page 1224, the third sentence, first paragraph, reads as follows:

> "The data as to the effect of forestation, soil absorption, and kindred methods of flood control are not at present susceptible of quantitative analysis."

Indicating our desire, as previously stated, to stimulate further research, the writer suggests the following substitute for the above quoted sentence:

> "Intensive study as to the effect of reforestation, soil absorption, and kindred methods of flood control is needed before quantitative analysis will be possible."

(4).—Page 1224, the second sentence, second paragraph, referring to frozen ground, reads as follows:

> "Such conditions prevent the utilization of forestation or ground storage."

As such conditions prevent such utilization at such times and such places, only, a more accurate statement is in the language recommended to be substituted for the above, *viz.*:

> "These conditions prevent at such times the utilization of reforestation or ground storage."

(5).—Page 1224, the second sentence, last paragraph, reads as follows:

> "There is a popular delusion that the same reservoir can be utilized simultaneously to reduce floods, increase the low-water discharge of a stream, and increase the water-power that can be developed therefrom, but ordinarily its utilization for any one of these purposes precludes its efficient use for either of the others."

As this indicates the kind of indirect adverse criticism which does not tell the whole story, the following is recommended as a substitute:

> "However, the same reservoir capacity cannot be used simultaneously to reduce floods, increase the low-water discharge of the stream, and increase the water-power that can be developed therefrom, but ordinarily, unless there be excess capacity, the use of the reservoir for any one of these purposes precludes its complete use for others."

(6).—The remainder of the paragraph just referred to comprises an entirely unnecessary and unfair discussion, purporting to show that reservoirs cannot be made useful for flood prevention, together with other purposes; whereas we know, on the contrary, that, notwithstanding all these statements, it is possible to operate reservoirs with several purposes in view, and that this is actually done in some of the great reservoir systems of Europe and America without conflict of interests. The following two sentences are recommended as a substitute:

> "It is evident, however, that such reduced efficiency for a combination of purposes does have some value, and the maximum efficiency for each purpose can be obtained by increasing the capacity sufficiently. The problem is an economic one, and much information regarding flood frequency, flood damage, reservoir operation, and comparative values of the several uses of storage is necessary before it can be satisfactorily solved in any case."

(7).—It is recommended that the first sentence in the first paragraph on page 1225, reading as follows, be omitted:

> "Your Committee, however, does not intend to condemn *in toto* the utilization of reservoirs for more than one purpose. In fact, it believes that"—

Such "damning with faint praise" is entirely unnecessary, and if such condemnation is not intended, it will be evident from the expressions, without saying so. The paragraph may well be introduced with the second sentence by omitting the first five words.

(8).—Page 1225, the third sentence, second paragraph, referring to the use of detention basins, reads as follows:

> "Flood control in such cases is only necessary during exceptional rains, and the basin itself may therefore be devoted to its normal uses most of the time, and only an easement on the lands occupied is necessary."

While it is understood that some agricultural pursuits can be carried on within and under the flow line of such detention basins, the use and occupancy of buildings, especially within their lower depth, likely to be filled to some extent each spring, cannot be permitted. Therefore the following is recommended as a more accurate statement, and as a substitute for the above quotation:

> "Complete utilization of the basin for such flood control is only necessary during exceptional rains and the land of the basin itself may therefore be devoted to its normal agricultural use most of the time and only the easement is necessary."

(9).—Page 1225, paragraph entitled (*c*): This seems to be a rather indefinite statement regarding barriers, and the following is recommended as a substitute:

> "Barriers erected across the valleys of tributary streams have been utilized to retain detritus and prevent its deposition in the main river. This has been found to be a suitable method in certain sections of our western country, where the wash of large amounts of material into the streams had disturbed the equilibrium and regimen. It is also used in Western Europe. Where conditions are favorable, this is a practical method of control, and its use will depend upon the slope of the stream and the character of the sediment being transported."

(10).—Page 1226, the last paragraph discusses the subject of increase of channel storage between levees. The writer believes that such storage is ineffective, in view of the fact that the potential storage over the surrounding country and overflowed lands is much greater than the volume confined between the levees. Therefore, considering any point on a stream below the length included within levees, the flood height is increased and not decreased by the introduction of the levees above. The writer, therefore, recommends that this paragraph be omitted.

(11).—For reasons which are apparent in the discussion hereinbefore given, the writer suggests that the four conclusions on page 1228 be amended to read as follows:

> "*Reforestation.*—The effects of forest growth in preventing erosion on hillsides are sufficient to justify reforestation for that purpose, but there has not been sufficient quantitative determination of its influence on stream flow which would justify a universal conclusion as to its employment as a method of flood control.
>
> "*Reservoirs and Detention Basins.*—At the head-waters of streams, storage reservoirs and detention basins can be successfully employed to reduce flood height. Local conditions will determine which method is preferable. The efficiency, however, of such agencies diminishes as the distance from them increases.
>
> "*Channel Enlargement, Cut-Offs, and Outlets.*—In clear-water streams, whose banks and beds are not subject to scour,

these agencies are useful. In streams carrying a large amount of sediment, whose banks are readily eroded, great care should be exercised in their employment, always bearing in mind the fact that on an alluvial stream best conditions follow its confinement to a single channel.

"*Levees.*—The influence of levees as a method of flood control becomes of prime importance on the lower alluvial reaches of long rivers and, upon certain streams, may afford the only method of flood control."

(12).—It is of little avail to submit such conclusions and recommendations without a suggestion as to some definite agency to carry on such work. Such constructive criticism will be helpful and should be presented. The writer therefore suggests the omission of the last paragraph on page 1228 and the substitution of the following, which should probably occur at the conclusion of the entire report.

"This subject is one of national importance, and while the work of State and local agencies should not be minimized, the necessity for the early establishment and unification, under National control, of systematic rainfall, run-off, and flood observations in greater detail than at present should be emphasized. Such authority should also suggest uniform methods of observation, co-ordinate information obtained, and preferably should have these duties as its main purpose, even though construction of important works for realization of results may be carried on at the same time by other agencies.

"The Committee desires to urge the great importance of the early establishment of such special agency, supported by adequate appropriations, for the purpose of studying stream regulation in its largest sense, and under whose direction all data shall be collated, according to uniform standards and systems, so that appropriate development of the science shall be made."

DISCUSSION

H. M. Eakin,* Esq. (by letter).—The Report of the Special Committee on Floods and Flood Prevention is a distinct disappointment to one interested in the general progress of applied science, who had expected the deliberations of the Committee to result in a constructive policy which would be clear of the inconclusive controversy that has heretofore hindered adequate progress in river improvement, and would, therefore, merit the endorsement of the Society and gain an effectively general support. Essentially, the report is an endorsement of specific projects of river improvement which are the subject of disagreement among scientists and engineers of standing. It also carries implications and statements that are not entirely sound. Mr. Eakin

The general spirit of the report implies that there exists an adequate, esoteric if not general, understanding of the scientific principles involved in river treatment, and that there is consequently only the need for such investigations as will furnish additional and standardized data as to the environment of their operation. The Committee evaluates with implied authority the relative merits of the various suggested methods of control, and shows its allegiance to current projects in statements that are not supported by fundamental scientific analysis.

The value of this implied authority may be indicated somewhat by an examination of the last paragraph under (*e*), pages 1226 and 1227. It is indirectly stated that the use of levees effects an increased channel storage that induces a lowering of flood levels. This is clearly impossible, since the channel storage given by levees is largely in space above the level which the same flood waters would assume if allowed to spread unhindered over the flood plain. Moreover, the confinement of floods between levees increases the facility of flow, so that discharge to successive reaches is progressively augmented above the normal as the effects of elimination of normal flood plain storage accumulate, and flood heights are correspondingly above the normal all the way to the sea. The increased flood height in the improved reach lowers the surface gradient for some distance up stream, decreasing velocities, and correspondingly increasing flood heights. The statement that levees reduce flood heights by increasing channel storage is obviously fallacious, yet it is advanced in the report as an important principle, generally overlooked, but fortunately recognized by the Committee.

The report fails of a much needed mission in not presenting clearly the present status of the science of streams. It does not indicate the scientific uncertainty as to the laws that underlie the basic problems of stream control, the reciprocal variability of discharge, grade, competence, capacity, load, torsional flow, the development, migration,

* Washington, D. C.

Mr. Eakin. and elimination of meanders, drainage distances, assortment of débris and extra-channel deposition—in short, the reciprocally determinative relation of erosional processes and physiographic forms, and the possibilities of selective artificial mutation and control of all these factors.

The problems of stream control reach out into many different branches of science—geology, physiography, meteorology, hydrology, agriculture, forestry, and engineering—and involve abstruse laws outside the scope of these special branches, and as yet imperfectly developed or only vaguely indicated. No individual or organization has as yet commanded such proficiency in all these sciences as to enable them to outline a programme of river treatment with dependable authority. Programmes such as have been outlined from the standpoint of a single science or profession have uniformly failed to withstand scientific scrutiny from other viewpoints. The result has been controversy, partisanship, and effectual inhibition of adequate progress toward the goal at which all honestly are aiming.

Progress of applied science demands, first of all, a dependable scientific authority. As regards stream control, we are apparently without such a basis. The Society probably could perform no greater service to the Profession and to the cause of river improvement and flood control than to devise a means of supplying this lack. The obvious recommendation is that a special organization be created with a personnel made up of variously trained scientists and engineers capable of assembling the pertinent matter of other sciences and of developing by appropriate researches an adequate, specific science of streams. The sound development of the science is prerequisite to its application. Only some such means as suggested can lift the matter of river improvement above the field of speculation and controversy, and make any considerable advance toward adequate achievement.

Mr. Hill. JOHN W. HILL,* M. AM. SOC. C. E. (by letter).—The frequency of disastrous floods in the Central and Western States during the past 12 or 13 years has directed serious attention to a condition affecting life and property which can no longer be tolerated. The periodical floods in the Mississippi and some other navigable rivers can usually be forecast as to time and river stage, and preparations can be made to discount more or less the resulting property damage; these floods or river rises are seldom attended by loss of life. There is no human foresight, however, which can predict the time of floods in the smaller non-navigable streams of the Middle West and western parts of the country, and it is with these that we are at present most concerned.

* Cincinnati, Ohio.

Floods in the large rivers of the Mississippi Basin are expected every year, although these are not always attended by great property loss or serious interference with business or transportation. Of course, there are no known means of preventing such floods. Rain and snow are bound to fall on the water-sheds, and the waters must flow away through the streams to the great reservoir in the ocean, to be caught up again into the clouds by evaporation, to make more rain and snow at a later date, and repeat the operation of precipitation, run-off, and stream flow, with frequent floods in the larger streams. Mr. Hill.

Measures to mitigate the destructive influence of great floods in the Mississippi River have been inaugurated and maintained by the States subject to damage, and by the Federal Government, Federal aid being given in the interest of navigation only.

A discussion of floods in the larger navigable rivers would have little bearing on the problem of flood prevention in the valleys of the smaller streams, and, in view of the appalling losses of life and property which have occurred from these in Ohio, Indiana, Kansas, California, and some other Central and Western States in recent years, the people directly affected by the floods have been compelled to insist on a remedy for an intolerable condition. One remedy would be to remove all habitations and perishable property permanently above the flood-plain; but this is not to be thought of as long as other avenues of escape from flood calamities are open. The vested interests in public and private property now lying below the flood-plain represent hundreds of millions of dollars in the States mentioned, and these must be preserved and protected, if individual property rights in these river valleys are to survive.

The frequency of destructive floods in the valleys of the smaller rivers and streams is often under-rated. Thus, in the Great Miami Valley in Ohio, instead of two or three great floods occurring within a century (a statement frequently made), at least five such floods have occurred within 110 years, namely in 1805, 1847, 1866, 1898, and 1913, the first and last dates marking the greatest floods.

Three of these floods have occurred within the writer's recollection. In 1866, while living in Dayton, Ohio, he saw the Miami River out of its banks and flowing several feet deep through the principal streets of that city. Marks made after the flood of 1913 indicated that, on Ludlow Street, south of Fifth Street, the flood of 1913 was about 5 or 6 ft. higher than that of 1866. In 1866, however, the city was much more open—with respect to bridges, buildings, and other obstructions—to the flow of water in the river channel and through the town, and the flood discharge for 1913 was not as much greater than that for 1866 as might be inferred from the difference in flood heights.

In 1866, Dayton had a population of 30 000, and had much unoccupied territory which is now solidly built up; no pretentious structures

Mr. Hill. had then been erected anywhere within the city limits; the river banks were not lined with dwellings or other buildings; and, when the river rose above the levees, the water found a reasonably unobstructed channel on the lowlands behind the levees on both sides of the river.

Prior to the flood of 1913, the river banks had been pre-empted and built on up to the levees, with a street or boulevard between; several reinforced concrete arch bridges with restricted waterways had been erected in the river channel, and, in many ways, the free flow of an overcharged river had been so much obstructed that the flood discharge of 1866 would have produced higher levels in the city streets had it occurred in 1913.

The flood of March, 1805, is well recorded for the locality of Hamilton, 35 miles from the mouth of the Miami River, by a local historian,* and this flood, in that early day, with no bridges across the river, an unobstructed channel, and few improvements of any kind on the river banks, reached a height as great as that of 1913. It is probable that the dense growth of timber and shrubbery along the river banks and on the bottom lands had some influence in impeding the free flow of the river and raising flood levels; and the existing "standing timber", or forests, may have had some further influence in retarding the run-off toward the stream. If these two conditions are supposed to have balanced, then it would seem that the run-off and stream discharge of the flood of 1805 may have been even larger in the vicinity of Hamilton than in the flood of 1913.

The recurrence of great floods in all streams can be surely expected, and the subject of the report of the Committee is the remedies to be sought to prevent future disasters from this source of danger.

The writer is in accord with the views expressed by Col. Townsend in a paper† read at St. Louis, Mo., in April, 1913, on the limited effect of storage reservoirs at head-waters on floods in the Ohio and Mississippi Rivers at Cairo, owing to the relatively small drainage area which such reservoirs can successfully control; and, as has often been stated, if such reservoirs should happen to be full, coincident with a great rainfall and run-off—which is a reasonable supposition during the latter part of the winter—it is difficult to conceive that they have any beneficial influence on the stream flow below, with the whole of the run-off from the controlled water-shed flowing over the spillways and into the discharge channels below the dams. The storage of great volumes of water from the higher portions of a water-shed simply for flood control can scarcely be looked on with favor. The use of the stored water for power, for regulation of stream flow, for irrigation, or for public or industrial water supply, would be sought and probably obtained after the construction of the reservoirs, if

* Mr. James T. McBride, 1831.

† "Flood Control of the Mississippi River," National Drainage Congress.

it had not been made a feature of the original enterprise. Unless the reservoirs were kept empty before a period of great or long-continued precipitation on the water-shed, they would fail to conserve and restrain the run-off for which they were planned; and, if kept empty in anticipation of heavy downpours, they would be useless for any other purpose than flood control. Mr. Hill.

The method pursued in Europe—to allow a certain depth at the top of the reservoir for flood control—is practicable, but, in all such instances, the water stored below the flood-plain is applied to some utilitarian purpose—water supply, power, etc. The writer does not recall any investigations showing the control of flood flow below the reservoirs accomplished by this method.

The considerable number of great reservoirs in Germany and France (as far as the writer is aware) have not been constructed only to prevent floods; they have been created for other and more sordid objects. Neither have any of the great reservoirs in India and the United States been built with flood control as the only object in view.

Concerning the influence of forests and reforestation on run-off and stream flow, during periods of heavy or continuous precipitation, the flood of March, 1805, in the Miami River, at Hamilton, is probably as great as any on record for that stream. At that date, the Miami Valley was largely a wilderness, with "standing timber" everywhere on the drainage area, except in the small clearings of the pioneers, and, therefore, one of two propositions must be accepted: either the rainfall exceeded that recorded for March 23d to 26th, 1913, or the forests had small or no retarding effect on the run-off.

No record of the rainfall for the 1805 flood is obtainable, and probably there was none, but the time of year and the height of the flood suggest conditions similar to those of March, 1913, when the rain fell continuously for 4 days, the ground and all vegetation was saturated, and a time was reached during the storm when the water was flowing off the water-shed at the same rate as it was falling from the clouds; and if, in 1805, the snow and ice were added to the precipitation, probably the run-off was greater than the rainfall.

The lack of rainfall records and data as to snow and ice conditions makes it impossible to analyze the 1805 flood; but, if the virgin forests did at that time retard the run-off and restrain the stream flow, then the rainfall was probably greater than any on record on this water-shed, and it is reasonable to suppose that it will be repeated in the future. No bridges spanned the Miami in 1805, nor were there any other artificial obstructions in the channel. River crossings were made at fords, by boats, or by swimming, according to the necessities and convenience of the pioneers.

The forests doubtless have some retarding effect on run-off, up to the point of saturation of the trees, undergrowth, and shrubbery,

Mr. Hill. and of the litter and humus on the forest floor. After this point has been reached, the soil in the forest is like that in the clearings, and, as all great floods follow long-continued or very heavy downpours, it would seem that, during the storm, the time must arrive when the retarding effect of the forest is nil.

In Ohio, Indiana, and Illinois there is very little "standing timber", the water-sheds of the streams are denuded, and, after saturation of the ground, the rainfall and run-off should be nearly, if not quite, equal. The retarding influence of ponds and pools on uneven ground, formed during storms, should not be neglected, but their effect on run-off and stream flow, after all, will be small.

The proposition of reforestation with a view to flood control in the large navigable streams may be practicable for the bare lands of the West, large areas of which are not under cultivation but are used only for grazing purposes; but it will be impracticable for the valuable agricultural lands in the valleys of rivers in the Central States. It is scarcely conceivable that a farmer will give up his land to reforestation for flood control, waiting 40 to 50 years until the growing timber on it becomes valuable, and meanwhile deriving very little or no return from the land; nor is it likely that any conservancy district could afford to purchase such lands and reforest them for this purpose. In any event, it is obvious that it would be many years before such reforesting could have much influence on the rate of run-off to the streams.

The writer agrees with Mr. Grunsky that cut-offs are sometimes advantageous, perhaps not in soil like that of the Mississippi channel below Cairo; but, when made in glacial drift formation, experience has shown that they will wear and accomplish good results in flood discharge. Assuming the natural materials of the river channel to be suitable for the maintenance of permanent cut-offs, there can be no doubt that, in a certain instance, where a river, along its meanders from its mouth to its head-waters, is now 170 miles long, and can be shortened by cut-offs to 130 miles, that, with the new slope and increased velocity, the flood-plain will be materially lowered everywhere in its length.

Where the bends of a river embrace a populous city, a cut-off would be prohibitive by reason of cost, and perhaps for other reasons, but through farm lands and outside built-up city limits, it can be adopted with advantage. Where the dredged material can be used in levees, the cost of cut-offs will not be prohibitive. Some cut-offs may require training walls, or spur dams, in order to maintain the alignment of the channel, and the materials for these can often be found on the ground.

With the materials before it from which to draw, it is to be regretted that the method of "flood prevention" by so-called "detention" or "dry" reservoirs has not been more fully developed. The literature

available to the writer shows few of such reservoirs, and none quite recent in date of construction, if the work now under way on the water-shed of the Little River in the southeastern part of Missouri, and a few reservoirs in prospect,* may be excepted. Mr. Hill.

Suppose a water-shed, having 75% of its area embraced in the drainage area of the small tributaries, generally of steeper slope than the rivers they feed, with channels often narrow and comparatively deep, often with rocky beds and conditions generally favoring the easy construction of check dams or barriers, and where the valley next to the river, as a rule, is wide and flat, and thoroughly developed, with many populous and prosperous cities and towns along the stream, the river very crooked, with the channel running through a drift formation ranging from builders' sand to large boulders, where cut-offs can frequently be made to straighten and shorten the channel, the stability of cut-offs and diversion channels in this instance having been proved by a test of more than 60 years: Is not this an instance where the retardation of flow—or, if desired, storage of water for flood protection—should be in the narrow valleys and ravines of the smaller tributaries, rather than in the main valley; and is not this also an instance where channel improvement will apply?

M. O. Leighton,† M. Am. Soc. C. E. (by letter).—The writer's impression, gained by reading this report, may be expressed substantially as follows: Here is a Committee of eminently qualified engineers —unquestionably as well qualified as could be selected from the Engineering Profession. To this Committee has been assigned one of the most important and difficult subjects in the engineering field. As the members of the Committee are eminent engineers, it follows naturally that they are very busy men. They realize, more acutely than any reader of their report can, that they have not had the time to give their subject the attention that it deserves. Having accepted appointment to this Committee, they do not wish to shirk the service involved. Therefore, they have made a safe and conservative report which, in all probability, each member regards as inadequate. Mr. Leighton.

Perhaps the writer's impressions are wrong, but, having had the benefit of acquaintance with some of the members of the Committee, and having, during past years, become familiar with the reputation and accomplishments of the others, he cannot avoid the conviction that this report on floods and flood prevention falls considerably short of the standards to which the Committee members have previously adhered. Such a result is not unprecedented in the lives of busy and well-qualified men.

The subject of floods and flood prevention is each year occupying more of the time and attention of National legislators. Among these

* Briefly mentioned in Appendix 5, of the "Report of the Pittsburgh Flood Commission".

† Washington, D. C.

Mr. Leighton.

legislators, doubt and diversity of opinion seem to be growing apace. One who has followed the events in Congress for the past ten years, and who has therefore acquired a mental picture of the legislative situation, must realize that the subject has not progressed to that point of finality with respect to fundamental facts and remedial measures which will insure a comprehensive legislative solution. The flood question crops out in debate on measures both relevant and irrelevant. The same old truths and the same old fallacies are printed each year in the *Congressional Record*. With admirable persistence, Senator Newlands of Nevada presents a great and comprehensive proposal that involves every feature of correction and improvement either closely or remotely related to stream flow and stream utilization. With equal admirable persistence, other Members of Congress advocate divers measures. The House of Representatives has recently created a new standing committee on floods and flood prevention, and it remains to be seen what the practical results of this new move will be. In connection with all this, two truths stand out prominently: First, that the Members of Congress who are expected to legislate wisely in this matter, and who, almost without exception, have no engineering qualifications, have the right to expect from the foremost civil engineering society of the country a definite and meritorious expression of opinion that will, in the end, define that which should be done. Second, it is the patriotic duty of every engineer, and certainly of this great National Society of Civil Engineers, to satisfy the reasonable expectations of the legislators.

It does not benefit any legislator to tell him about fundamental principles or to give him a summary of the things which cannot or which should not be done. The legislator is looking for a concrete proposal that will stand the test of analysis. A legislator who may be utterly incapable of formulating a proposal may be quite successful in analyzing, criticising, and pointing out, the weak spots in a proposal submitted to him by qualified engineers. Let us not deceive ourselves about this matter. If the flood problem is to receive final and successful legislative treatment, it will be only after the engineers of the country have come forward with a definite legislative measure, based on physical facts and well-attested engineering principles. This statement may do violence to the sacred tenets of the American Society of Civil Engineers. The writer can already hear the deprecatory expressions of many members with respect to the Society's participation in legislative matters and advocacy of any particular proposal. There is no doubt that such an attitude is comfortable, dignified, and utterly safe, but the fact remains that, if the Society expects to see the consummation of a successful solution of this problem, it must do the useful and necessary thing, rather than the dignified and comfortable one.

Mr. Leighton.

There is no trade or profession in the United States in which one may find such persistent and general expressions of discontent over the inadequacy of legislation as in the Engineering Profession. It is also fair to say that, in the opinion of the writer, there is no profession in which such expressions are so intelligently and considerately given. On the other hand, it is the writer's observation that there is no class of complaining citizens which so consistently fails to come to the aid of legislators with definite concrete advice. If the members of the Society dissent from this statement, let them ask the legislators.

From the foregoing, it will be seen that the writer criticises the Flood Committee's report because it leaves the country and the country's legislators precisely where they were before the Committee undertook its labors. If a traveler who has lost his bearings and has become confused amid a maze of divergent paths, asks the route to his destination, he is not benefited by a reply which says in effect: "Friend, you must take the proper road that leads to your destination; if you go by the wrong road, you will probably not arrive." The writer is very glad that the Committee has been continued, and he submits the foregoing humble suggestions in the hope that they may have some bearing with respect to the future report which it is assumed that the Committee will make.

With respect to the actual findings of the report, the writer agrees in the main with the minority views of Mr. Knowles. Though it is true that many of the sentences which Mr. Knowles proposes in substitution are, in the opinion of the writer, merely improvements in English composition, they do add to the judicial temperament of the report.

The Committee's conclusions (page 1228) with respect to reforestation may or may not be correct. The writer believes that they are incorrect, but, assuming their entire correctness, it is apparent, to the unbiased readers to whom the writer has submitted this paragraph, that the Committee has either consciously or unconsciously made a seemingly judicial statement which, among legal brethren, would be regarded as a very adroit special plea. The half-truth expressed in this short paragraph does more to convince the reader against reforestation than would the plain blunt statement that forest growth has no effect whatsoever. Paraphrasing the concluding words of the statement, the Committee, with equal truth, could have said: "But there has been no quantitative determination of its influence on stream flow which would justify its exclusion as a method of flood prevention." The writer is familiar with intensive investigations, the results of which have unfortunately never been published, which show mathematically that forest growth does, under certain conditions like those prevailing

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in the White Mountain region of New Hampshire, markedly reduce or meliorate flood extremes.

The Committee has disposed of reservoirs and detention basins as factors in flood prevention by a few short, obvious, and rarely disputed statements. They give the reader the impression that the Committee, on the whole, is doubtful concerning reservoir efficiency; yet, after one thoroughly analyzes the section, he may be able to agree with every statement and still remain a pronounced advocate of reservoirs. This advocacy does not commit any one to the reservoir theory under all circumstances and conditions. It merely commits the believer in reservoirs to the old common law doctrine that every cause is entitled to its day in court. An apparently reluctant admission, such as that made by the Committee, that reservoirs may be beneficial under some conditions, cannot act as a counter-weight to a series of objections and adverse implications which may or may not apply in any particular case. There is, to say the least, a lack of fairness in a pronouncement which gives qualified approval to reservoirs and then cites a series of adverse conditions without also giving to them equal and well-merited qualifications. Altogether, Mr. Knowles' expression "damning with faint praise" is applicable and well-chosen.

The Committee has wisely set forth the deficient and unsatisfactory condition of available stream flow data. It might, with equal wisdom, have set forth the unsatisfactory condition of our knowledge with respect to detailed topography, reservoir sites, and storage management. These deficiencies have been apparent for a score of years. What has the American Society of Civil Engineers done to correct that condition? Federal and State appropriations for such investigations have either been lacking or inadequate, but that is a natural consequence of the lack of information on the part of the people. It cannot fairly be expected that any considerable body of voting citizens is going to become enthusiastic over a fundamental engineering necessity which they do not understand. Who is there to explain and instruct, except the engineers of the country? The lack of appropriations is not entirely responsible for the paucity of data. Some of the blame can be traced to the inefficient expenditure of the money that has been provided. In making this statement, the writer does not intend to reflect on any particular State or Government Bureau, and, if he were disposed to make comparisons, there is one particular Government Bureau which he would exclude from this category, because it has come pretty close to making each dollar produce results to the value of two dollars. The fact remains, however, that engineers, and especially those in official life, have not expended their efforts and appropriations along the most purposeful lines. Looking back over the history of river hydraulics in America, we commonly start with the work of Humphreys and Abbot on the Lower Mississippi.

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That classic investigation had one bad result. The engineers accepted it as a finality rather than a most remarkable beginning. Perusal of the reports of investigations that took place during the 25 or 30 years subsequent to the Mississippi investigation will show that the efforts were largely devoted to an attempt to prove how eternally right or how grievously wrong were Humphreys and Abbot. Had our work on river hydraulics during those years been a forward movement, rather than a *post mortem,* our present Committee on Floods and Flood Prevention would undoubtedly have found itself less embarrassed by a lack of fundamental data. A recital of past errors is useful only in so far as it may serve as a guide to future conduct. This Flood Committee could, if it desired, do more than merely report regretfully the deficiencies in our fundamental facts. It could, if it chose, become the fountain head of a campaign by which a beneficial change of conditions could be brought about.

Mr. Babb.

CYRUS C. BABB,* M. AM. SOC. C. E. (by letter).—This report is a good, concise statement of our present knowledge regarding floods and flood prevention methods. It brings out forcibly the lack of adequate engineering data on this extremely important subject.

It may be that the data collected by the Special Committee are too voluminous for detailed tabulation in a comprehensive report, but it would seem as though a bibliography or reference table of such data, arranged by drainage areas where possible, might be published with profit to the Society.

It seems to the writer that the Minority Report by Mr. Knowles contains, in a few cases, better expressions for the same ideas than those submitted by the majority. The second heading is a more fair presentation, giving the advocates of reforestation an even standing with the opponents.

The writer is in hearty accord with Mr. Knowles' twelfth heading and of placing it at the end of the report. It would seem as though the U. S. Geological Survey, through its Water Resources Division, was the proper bureau to carry on the investigation. This Division might well be elevated to the rank of an independent bureau, as was done a few years ago in the case of the U. S. Reclamation Service.

Mr. Grant.

KENNETH C. GRANT,† M. AM. SOC. C. E. (by letter).—Experience in connection with flood prevention legislation and investigations has strongly impressed on the writer the value of a carefully prepared report on the general subject of floods and flood prevention, such as has recently been submitted by the Special Committee. Such a report, to measure up to its greatest possible usefulness, should deal with every possible phase of the subject, and should set forth clearly the

* Rhodhiss, N. C.

† Washington, D. C.

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advantages and disadvantages of every known method of solving the flood problem. It is particularly gratifying, therefore, that the Committee has been continued, with opportunity for written discussion by the membership at large, in order that the Committee may have at its disposal the complete information necessary to make its final report the most comprehensive and authoritative document on the subject to date.

The writer is in accord with the position taken by Mr. Knowles in the second paragraph of his minority report. The report of the Committee will do the most good if it presents both sides of disputed questions, with the reasons for and against, and points out, wherever possible, the line of investigation needed to throw additional light on the problem.

A case in point is the opinion of the Committee regarding the use of reservoirs for more than one purpose. Engineers in America have not had sufficient experience with intensive, extended rainfall and stream-flow studies to enable them to state conclusively that reservoirs cannot be manipulated in such a way by flood-warning stations that part, at least, of their capacity can be used for more than one purpose The writer has examined a number of large reservoirs which are being operated successfully in this manner in Germany and Austria. Such combined use is especially feasible in regions where floods are confined to one season of the year. Rigid supervision of the construction, and especially of the operation, of such reservoirs by some competent authority is, of course, absolutely essential.

Another case is the treatment by the Committee of the subject of reforestation. The writer would be glad to see the report point out that sufficient data are lacking to prove whether reforestation does or does not regulate stream flow sufficiently to justify its use for that purpose; that if it does have a tendency to reduce flood flow, the partial prevention of floods by such a means would be a slow process, not feasible in any situation where immunity from floods was required at once, although it should be carried out to whatever extent feasible, as a contributory benefit to stream regimen; that reforestation, by preventing erosion, does prevent the filling up of regulated channels, reservoirs, and detention basins, and thus indirectly does assist in flood prevention; that in many cases, even if it were proven effectual, it could not be applied, owing to the value of the lands in the drainage area for other purposes; in short, giving the results of the findings of the Committee on both sides of the question.

An interesting quantitative analysis of the effect of forests on flood run-off was worked up by the French Engineers, in studying plans for flood prevention on the Seine, after the great flood of 1910. Their figure for the depth of rainfall absorbed by forest cover was about 0.41 in., to which they added about three times this quantity for the

rainfall used up in 10 days by transpiration. With these figures, the action of 10 000 acres of forest cover in preventing run-off is equivalent in 10 days to a storage reservoir capacity of about 60 000 000 cu. ft. Mr. Grant.

The cost of reforesting 395 000 acres in the Seine Basin, including the cost of land, planting, and care, was estimated at $84 400 000. In 10 days this would reduce the flood volume by 2 370 000 000 cu. ft. The cost per million cubic feet of flood water retained by forest cover would thus have been $35 600, about half of which would have been returned in 60 years from the main cut of the timber. This was too high a figure to be considered, especially as the total volume of the 1910 flood at Paris during the maximum 10 days was 70 600 000 000 cu. ft., and during the entire flood was 247 100 000 000 cu. ft.

The French engineers, therefore, recommended that the Government should not take up the reforesting of the Seine Basin directly; especially as they considered that the action of heavy forest cover in retarding run-off disappears almost entirely in the great rainfalls that cause the Seine floods; and also considering the rich agricultural lands that would be taken up for forests. Their decision was that the Government should encourage all works of forest culture, should preserve intact the dense forests then existing, by improving the methods of cutting, and should lighten taxes on forest property.

Popular discussion of the causes of floods has laid so much emphasis on the influence of drainage that it would be helpful if the report discussed this subject. In some cases, properly executed drainage works increase the capacity of the drained lands for retaining water, and improve the soil cover. Furthermore, when irrigation and drainage are combined, increased retention is obtained. In any event, the needs of the improvement of cultivation in most cases notably outweigh any damages resulting therefrom.

As a means of retarding run-off, decreasing erosion, and increasing seepage in mountainous and hilly country, horizontal ditches have frequently been discussed by European engineers, but their use has been recommended only in rare cases. Suitable places for utilizing this method are not available to any great extent. They interfere with the use of the ground for other purposes. The cost of construction and maintenance is also too great. If 4 in. of rainfall are to be held back, these ditches must be about 3 ft. deep and 3 ft. wide, and require from one-sixth to one-fourth of the total land surface thus treated. At 30 ft. apart, about 1 450 lin. ft. would be required per acre, or about 928 000 lin. ft. per sq. mile. The cost would be from $13 000 to $26 000 per sq. mile. Such works favor irrigation by keeping up a dampness in the soil and increasing the supply of groundwater. An advantage of this method is that it may easily be carried out by private individuals. It is practised by the farmers of the Cevennes Mountains, in France, and in Tunis.

Mr. Grant.

The last paragraph under (*e*) *Levees*, on page 1226, may be misleading. Unless its exact meaning is studied carefully, it may give the impression that the decrease in valley storage, by confining the water between the levees, had been lost sight of. The comparison between possible methods of protection may be made clearer if it is granted, in the first place, that the lands behind the levees must be given the same protection by each method considered.

Levees of suitable size will protect the low lands behind them. The volume of water stored between the levees will be very great. To give the same protection to these lands without levees would require that this water be stored elsewhere.

It might be stored in a channel excavated to give the same cross-sectional area as that provided between the levees. This would generally be much more costly to build and maintain than the levees. There might be serious complications at the mouths of the tributaries and at the mouth of the river. The ground-water conditions might be very harmfully affected. The added cost at bridges would be very great.

It might be stored on the drainage area above, if sites could be found. The total capacity of such storage basins would have to be much greater than the storage space between the levees, in order to take care of variations in distribution of flood rainfall, and to make up for the reduction in the effect of storage that takes place in passing down stream.

One statement in the paragraph alluded to may be questioned, unless the writer has not fully gathered its meaning, namely, at the end of the first sentence: "the resultant reduction of flood height from this cause". It is unquestionably true that the artificial storage between the levees is not so great as the natural storage that was available over the lowlands along the river before the levees were built, and that this reduction in storage raises flood heights. At the same time, the contraction of the flood flow between levees raises flood heights at and up stream from the stretch of river that has been leveed. When adequate levees are built to above this increased flood height, and properly protected and maintained, they afford one of the surest and safest means of flood protection.

In view of the action of levees in reducing natural storage and thus increasing flood heights, especially in the river below, proper precautions should be urged that such improvements be not made on any reach of a river until adequate provisions have been made at other points to take care of the changed conditions of flood flow. There are points here which can well be considered in discussion of the Report of the Special Committee on A National Water Law.*

* *Proceedings*, Am. Soc. C. E., for December, 1915, pp. 2747–51.

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The French water laws specifically name what rivers or stretches of rivers may not be leveed to above maximum flood stage. On the Loire, wide overflow spillways have been provided on the levees to allow temporary storage in the lowlands behind them during extreme floods. It has for years been a subject of discussion and investigation in Austria and Germany as to whether the levees could not be altered so as to keep out only the lower floods of the growing season, and allow the great winter and spring floods to overtop the levees and spread over the lowlands. The reduction of flood heights thus obtainable is one argument in favor of such treatment; another is the fertilizing action of these silt-laden waters on the lowlands behind the levees, and great stress is laid on this feature in all reports on the subject. The overflow areas in the German river valleys are to-day only about one-fourth to one-fifth of their natural and original extent.

The natural storage effect on the overflowed lowlands along a river may be very considerable, and may have an appreciable effect in lowering flood heights. In the Miami Valley above Dayton, the valley storage in the 1913 flood amounted to about 300 000 acre-ft., or 32% of the total run-off from the 2 500 sq. miles above Dayton during the maximum 3 days of the flood.

On some rivers a disadvantage of building levees reaching to above high water is that the lowlands behind the levees are no longer raised by deposits, and sometimes even sink lower; and the river itself may gradually raise its bed by deposits in its middle and lower course, where the slope is flat. These lowlands can even gradually become swampy, and the pumps formerly used for freeing them of water in the spring only will have to be kept at work longer. This is especially to be feared in the lowlands on low deltas, which are gradually extended into the sea by deposits of silt, so that the raising of the water surface in the river is necessary, in order to create the fall required for discharge through the longer channel.

On the other hand, the narrowing of the channel by levees, by increasing the velocity, is oftentimes not only favorable for transportation of silt, but indeed forms the only available means for preventing a raising of the river bed. Without the levees, the banks and land immediately bordering them would be gradually raised, and lowlands farther back would remain low; and sometimes rivers would break through and seek a new channel in the lowlands, with disastrous results. This danger is especially great in the upper courses of rivers, where the slope is steeper. The levees reaching to above high water there form the only effective means of preventing this.

In channel improvement by deepening and straightening, due consideration should be given to the effect on ground-water conditions. On the Weser River, in Germany, agricultural interests have required

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large expenditures in order to restore the ground-water conditions that obtained before the channel was deepened.

It is important, in connection with later plans for channel improvement, that some suitable agency be authorized to exercise control over the placing of obstructions in and along streams. The cost of adequate channel improvement in some localities is greatly increased, and sometimes made prohibitive, by structures that should never have been allowed to be built in their present location or to their present dimensions.

On some rivers, particularly where melting first takes place on the upper river, as on the Oder and Weichsel in Germany, the formation of ice jams is a frequent cause of floods. The design of improved channels on such rivers should take this into consideration. In special cases ice-breaking steamers may be found necessary to prepare for spring freshets from the upper river.

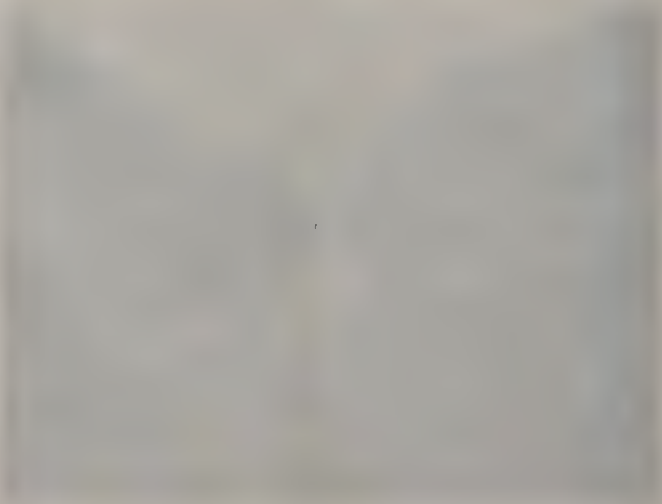
Detention basins as a means of flood control are stated by the report to be successful in certain localities. This method, as yet little used in America, should perhaps be discussed in more detail. There are localities where this is the only kind of storage feasible. The agricultural lands may be too valuable to be dedicated to permanent storage. Floods may not be confined to any one time of year, so that where the drainage areas controlled are small and floods collect too rapidly to permit the emptying of a partly full reservoir in time to provide the required flood storage space, all the capacity intended for flood control must be kept empty, and ready for immediate use. In such cases, if the entire capacity available is required for flood storage, a "dry" reservoir is the only type feasible. Detention basins, as compared with reservoirs, have the disadvantage of wasting the stored flood water; and in some localities this may be a very important consideration; but, where the foregoing conditions prevail, and where flood control is a vital enough issue to warrant their construction, they will sometimes be found the best solution of the problem.

Where assurance can be had of competent and permanent supervision and maintenance, detention basins can be made much more effective by providing artificial control. Where there are no gates in the outlet conduits, they must be designed so that the maximum flood will just reach the spillway when the inflow has fallen to the same rate as the outflow. This, however, gives a much less degree of control over smaller floods than could be obtained by the manipulation of gates in the outlet conduits.

The great advantage of detention basins is that they do not remove the land in the basins from cultivation. In the Miami Valley, where the conditions for this method of flood control are particularly favorable, the total area flooded in the five proposed basins, with a flood like that of 1913, would be 29 850 acres. More than 60% of this same land (18 700 acres) was actually flooded in 1913. After such a flood all the

FIG. 1.—PINAY DAM ON LOIRE RIVER ABOUT 7 MILES ABOVE ROANNE, FRANCE.

FIG. 2.—TYPICAL EXAMPLE OF WORK OF REFORESTATION AND MOUNTAIN-TORRENT CONTROL ON HEAD-WATERS OF ELBE RIVER.



reservoirs except one would empty again in a week, and that one would take about 3 weeks. In smaller floods, they would flood less land and empty much more rapidly; and, even in extreme floods, a large part of the area would be flooded for only a few days. The area actually flooded in 1913 in the Miami Valley above Hamilton was 63 000 acres, or more than twice as much as would have been flooded in the detention basins if they had been in operation. Of this flooded area, 13 000 acres were in cities and towns. Mr. Grant.

Fig. 1 shows the Pinay Dam, on the Loire River, about 7 miles above the City of Roanne in France. This is one of the most interesting examples of a detention basin in Europe. It was built by the French Government in 1711, and has since repeatedly saved the city from destruction.

It should be pointed out, in discussing reservoirs, detention basins, and levees or other channel improvements, that in many cases no one method may provide an adequate solution of the flood problem, but the best plan will consist of a combination of two or more methods. It may be found unfeasible or uneconomical to reduce flood flow to existing channel capacities by reservoir or detention-basin control. It may be found equally unfeasible or uneconomical to provide an adequate channel to accommodate maximum floods. Protection by one method is best carried out up to the point where the remaining necessary protection is cheaper by some other method.

In discussing the function of barriers in mountain streams, their action in increasing soil absorption should be included. Under certain conditions, as in Los Angeles County, California, this may be very considerable, and may bear an important relation to other phases of the water problem.

The important action of barriers in holding back sand, gravel, and boulders from improved channels and reservoirs below should be brought out more definitely. In some of the comprehensive river improvements of Europe, reforestation and *Wildbachverbauung* form an integral part of the reservoir systems and channel improvements, and are regarded as essential to the maintenance of such works on the lower streams. The River Regulation Commission of Bohemia has been carrying on a remarkably well co-ordinated work of this character on the upper Elbe, including reforestation, barriers on mountain torrents, flood-control reservoirs and channel improvements.* Fig. 2 shows a typical example of the work of reforestation and mountain torrent control on the head-waters of the Elbe.

The report makes no reference to the method of reducing floods by spreading over permeable strata. It would be helpful if it would take up the advantages and disadvantages of this method and its re-

* *Engineering News*, October 15th, 1914.

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lation to underground storage, and make some reference to localities where it has been or might be applied successfully.

Desynchronization of tributary flood peaks is an effective means of flood prevention in particular cases. It will generally be insufficient by itself, but will lessen the work to be done in connection with other methods with which it may be combined. In mountain streams, tributary flood peaks that reach the main stream at or after the passage of the flood peak of the latter can be retarded further by barriers or small detention basins, and those that arrive early can be accelerated by the improvement of their channels.

The same process on a large scale can sometimes be carried on with the larger streams, farther down. A flood wave can be made to travel twice as fast without any increase in the average velocity of the water, by establishing the proper cross-section. It can be demonstrated that, with a parabolic cross-section the speed of propagation is approximately four-thirds of the average velocity. Tributaries that deliver their flood peaks early can thus be further accelerated without any increase in erosive action. Other slower tributaries can be further retarded, where feasible, by reservoirs or detention basins, or by wide overflow areas along the banks.

Obviously, in solving the flood problem of any river basin, it is very important to avoid the more perfect synchronization of tributary flood peaks that might be brought about by reversing the treatment just outlined. It would be dangerous, for example, to construct detention basins or provide overflow areas on tributaries delivering their flood peaks early, unless the basins were large enough and the outlets small enough to reduce the flood wave sufficiently to preclude any possibility of aggravating the flood conditions on the main stream. In the same way, as a general principle, it would be unwise to speed up the flood wave on a slow tributary.

The report might well include some reference to flood prediction and its value in preventing or reducing flood damage. This is a measure of relief that can be applied at once, pending the construction of flood prevention works. It has been very thoroughly worked out in some European countries, but has not yet been developed to its full possibilities in the United States.

The figure quoted in the report for the flood losses in 1913 in the Ohio Valley is much too small. The direct property loss in the Miami Valley alone was about $70 000 000, and to include the loss through depreciation would greatly increase this figure.

The writer joins most heartily with the Committee when it urges more thorough, intensive, and co-ordinated studies of matters relating to flood control. Such investigations should include all data necessary for the solution of problems of water conservation and stream control, in order that flood prevention works may be co-ordinated with

other uses of the streams. Enormous appropriations for river regulation and water conservation have been urged before Congress for some years. If these appropriations were to be made to-day we would not have the data with which to design, intelligently and economically, the works to carry out the purposes thereof. The proponents of such measures would do far more good in the development of the country's water resources if they fathered successfully a bill appropriating several million dollars a year for the use of the present Government agencies now struggling to collect data with pitifully inadequate funds for operation. Mr. Grant.

In nearly every problem of river control attacked by the engineer in America, he finds himself confronted with almost a blank wall of lack of data, and is obliged to make safe assumptions that greatly increase the cost of the work. River gauging stations are too widely scattered, and lack of funds prevents their efficient operation. At many of them the gauge is read but once daily, so that there is no complete record of flood peaks, as they frequently occur between readings. The number of rainfall stations should be very largely increased, and hourly readings should be taken during rain storms. It is not possible, with the present methods of collecting and recording rainfall data in the United States, to determine successfully the relation between rainfall and run-off.

The writer would be glad to see the report make definite recommendations to remedy this condition.

B. F. Groat,* M. Am. Soc. C. E. (by letter).—A reading of the Report of the Special Committee on Floods and Flood Prevention, the Minority Report by Morris Knowles, M. Am. Soc. C. E., and the subsequent discussions by Messrs. Eakin and Hill, discloses the fact that the report does not seem to cover all the ground expected, and that possibly suggestions in more definite language as to general policies and methods would be acceptable. Mr. Groat.

It appears to the writer that the whole question of flood control is necessarily indefinite at present and probably will so continue for a long time. The indefiniteness and uncertainty of such a report, therefore, is a natural and certain consequence of the circumstances and present state of knowledge of the subject, rather than a result of any neglect or lack of thoroughness on the part of the Committee. Under the circumstances, one can scarcely expect a committee to devise revolutionary constructive policies, or express conclusive opinions, concerning a question which has perplexed all the rest of the world since the idea of national flood control developed and raised itself from a special to a general engineering problem, intimately related to that of national conservation. At least, this is the opinion of the writer, who is more or less of an exoteric with regard to this particular subject.

* Pittsburgh, Pa.

Mr. Groat. Mr. Knowles' objections to the report appear to be based largely on phraseology which he considers to be single-viewed, and to partake, so to speak, of the prejudice of the Committee. The writer believes that a committee report is no place for the individual opinions of the members, except when expressly stated to be such, and that where controversial matter is involved, the evidence on both sides should have equal publicity. Engineering questions are not to be settled as are questions of law—by arbitrary rule—but they should lead to the determination of a rule, or procedure, which shall meet with either general approval or the specific approval of the Engineering Profession.

As regards breadth of view, there does not appear to be much room for exception, and it must be observed that, with the amendments suggested, the report would be even more indefinite than without, as should be expected in the case of this subject, except that Mr. Knowles does ask for additional kinds of data, as well as an increase in the quantity of such data as are now regularly observed by various agencies. The Government agencies might increase the variety by adding such information as data on erosion and transportation, precipitation for various altitudes at the same place, and for different places and circumstances, especially in higher altitudes and in sparsely settled districts. Some systematic method might be recommended for the study of forest reaction.

The most important policy suggested by Mr. Knowles is that briefly outlined in the last paragraph on page 1234. It appears to be admitted by all that the whole question of floods and flood prevention is of vital importance, and demands special efforts and expenditures commensurate with the difficulties encountered. Why not, therefore, have a "special agency, supported by adequate appropriations, for the purpose of studying stream regulation in its largest sense?"

Mr. Chittenden. H. M. Chittenden,* M. Am. Soc. C. E. (by letter).—The writer does not fully understand the purpose of a report of this character, whether it is simply to remain as the findings of the Committee, or, after discussion, to be adopted, with possible modification, as an official expression by the Society. There are certain reasons why the latter course, if practical unanimity could be arrived at, would seem, at the present time, to be particularly desirable. From this point of view, the writer will contribute his mite toward the further elucidation of certain features of the Committee's able treatment of the subject.

Terminology.—First, as to terminology and verbiage: Ought not "control" to replace "prevention" in the title? Prevention is not broad enough. Levees, for example, do not prevent floods (they actually increase gauge height and discharge), but they control them. Reservoirs prevent, or tend to prevent, floods. The comprehensive word is "control," with a logical subdivision into measures for flood forecast or

* Gen. Chittenden, who lived at Seattle, Wash., died since writing this discussion.

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warning, measures of prevention, and measures of protection. The distinction between prevention and protection is well brought out in the report of the Pittsburgh Flood Commission.

On page 1219, under "Suggested Methods", Subdivision (*a*), on the first line, should not "rainfall" be "run-off"? Man cannot reduce "extreme variations in rainfall", as the Committee correctly states in the last paragraph on this page, but he may reduce extreme variations in the results of rainfall.

Would it not be well for the Committee to recommend the definite adoption of one or the other of the equivalent terms, "drainage area", and "water-shed". The term water-shed was formerly used, and occasionally is still used, to signifiy "divide"; but its use is now almost universally confined to a designation of the territory drained by the particular stream under consideration. Probably no confusion results from the use of both terms, but simplicity of terminology is always desirable. The writer favors the term water-shed.

Several designations have been applied recently to the comparatively novel system of flood prevention which is the distinguishing feature of the great flood-control project being worked up by the Miami Conservancy District. These include "dry reservoirs", "detention reservoirs", "detention basins", and "retarding basins". When the writer was preparing his paper on Flood Control for the recent International Engineering Congress in San Francisco, he gave this matter some little consideration. Inasmuch as reservoirs may properly be classed in two distinct types, dependent on their purpose and functioning—those designed to store water for future use, and those designed to detain it temporarily for flood prevention only—it seemed wholly logical to use "reservoir" (Latin, *re servare,* to hold back) as the generic term, so to speak, and "storage" and "detention" as specific designations. It was for this reason that he felt particular about adopting "reservoir" as against "basin". The Miami Conservancy District has definitely adopted "retarding basin", and the prominence of this example ought, perhaps, to settle the question.

Effect of Human Occupancy of Soil.—The writer feels that the Committee might advantageously have been more definite and positive in its discussion of this subject, particularly as it relates to the value of reforestation. For reasons stated later, a definite pronouncement on this subject by a professional body like this Society would have a most important influence. Doubtless, there is some reluctance to assume a positive attitude on a question which, as Mr. Knowles says, is "one of the most controversial subjects upon which engineering and scientific minds have been recently engaged." It may be questioned, however, whether to-day there is much controversy among well-informed engineers as to the essential principles involved. Is not the consensus of professional opinion that, in times of excessive rainfall, in which

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really great floods always originate, the restraining effect of forest cover, whatever it may be in ordinary storms, becomes exhausted and is of no effect? This is the official opinion in France, even in forestry circles, as is shown by the following extract from a report by the Director of the French Forestry Service on the 1910 flood of the Seine:

"In the exceptional circumstances which always produce the floods of the Seine, the action of forests, whether evergreen or deciduous, becomes for the moment negligible in its effect on the volume of water carried by the river."*

Engineers are apparently agreed on this essential point. In formulating plans for flood control, they would not dare to reduce in any degree the effectiveness of direct measures, such as reservoirs, levees, etc., by any possible measures of reforestation. The Committee brings out this point in the first paragraph of page 1224 of its report, but it weakens its position by holding out the questionable hope, implied in the words "not at present susceptible", etc., that the future may demonstrate a value in forest cover not at present apparent. Mr. Knowles, on page 1231, is outspoken in the expectation that "quantitative determination" of the effects of forests on run-off is a possibility of the future. The writer dissents wholly from that opinion. The difficulties in the way of any such result are practically insuperable. A determination for one situation, or season, or storm, would never apply reliably to any other. The modifying influences are infinite in detail, and the utmost that we can hope for is a gross integration of results, as shown in the records of stream flow. This has already been carried on in Europe for more than a hundred years, with a wholly negative result so far as the favorable influence of forests in reducing run-off is concerned. It seems very desirable, therefore, that a report of this character should not hold out any false encouragement.

At this point, it may be well to consider a feature of the subject not mentioned in the report, yet a most important one, and one concerning which there may be differences of opinion, even among engineers. Reference is made to the influence of drainage on flood run-off. The obvious and plausible inference is that drainage—roads, pavements, sewerage, farm ditches, tile drains, etc.—expedites the run-off and fills up the streams more rapidly than formerly; but there are very strong counter forces which operate to retard run-off, particularly from originally wet and marshy soil and from swamp areas. This is the creation of ground storage where none existed before. The very

* "Mais, dans les circonstances exceptionelles dans lesquelles se produisent toujours les crues de la Seine, l'action des forêts, fussent-elles résineuses ou feuillées, devient momentanément négligible sur les volumes des eaux roulées par le fleuve. Les pluies intenses et persistantes de la seconde quinzaine de janvier dernier, ayant saturé le sol forestier, commes les autres terrains, les eaux tombées à la surface sont toutes allées aux thalwegs." M. Daubrée, Conseilleur d'Etat, Directeur Général des Eaux et Forêts. Report of First Commission on Flood of 1910 in the Seine, p. 515.

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purpose of drainage is to get water out of the ground; but when so gotten out, space is left for other water. The most effective and instantaneous of all reservoirs, so far as flood control is concerned, is the top 18 in. of soil, depending on the extent to which it is saturated. A swamp soil undrained is generally thoroughly saturated. It comes next to an actual body of water in that respect. The only, or the main, storage available is surface storage due to the conformation of the basin. Drainage takes the water out of the swamp soil and creates vast storage space where before none existed. This effect is fully recognized by the French. It is, of course, greatest with summer or early fall storms. Like all surface storage, it may become greatly reduced or completely exhausted by the long rains of winter.

With the utmost that can be done in attempting to evaluate the influences of man's occupancy of the soil in modifying the effects of soil cover and absorption, we find it impossible to say on which side the balance lies. It is probably not important, either one way or the other, and, in any event, becomes wholly obliterated in those excessive rainfalls which alone produce great floods. This conclusion harmonizes both with theory and stream-flow records, and should be accepted as definite and final. The writer emphasizes it, not because the popular view to the contrary influences the engineer to any great extent, but because it does influence the legislator to whom the engineer often has to look for funds, and it thus, in the words of the income tax law, "deducts" from his power of effective action "at the source". How great is the importance of taking a definite stand on this question may be judged when a distinguished United States Senator, who pretends to some acquaintance with these subjects, and whose opinions carry weight in legislation concerning them, is capable of delivering, in apparent soberness and good faith, an utterance like the following:

"We have been destroying our forests, great natural reservoirs of moisture into which the waters fall from the heavens, and where they are stored in the leaves and in the loose soil, and drunk up by the thirsty roots of the trees and vegetation, and thence the surplus gradually makes its way to the creeks and tributaries of our rivers, and the water which used to be absorbed by these forests is now hurried on into the creeks and rivers."*

* Hon. Francis G. Newlands, in the United States Senate, Feb. 21st, 1916. It is interesting to apply this to a particular case. In the country around Dayton, Ohio, in March, 1913, after heavy winter rains had quenched to satiety any thirst from which tree or other roots might have been suffering, there fell, in a period of 4 consecutive days, between 9 and 10 in. of rain, or more than one-fourth of the average for an entire year. Assuming that the virgin forest had still covered the country, what percentage of this deluge does Senator Newlands imagine would have been "drunk up by the thirsty roots?" And what is Senator Newlands' warrant for calling the soil of the forest "loose"? Has he ever noted a great tree which has been overturned by the wind and has carried up with its roots perhaps 100 sq. ft. of soil so densely compacted that it resists for years exposure to the elements? The truth is that virgin forest soil, undisturbed for ages, is one of the most compact of soils, whereas cultivated soil, ploughed and harrowed and worked over, and full of perennially decaying roots, has the maximum absorptive capacity of which a given quality of soil is susceptible.

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If the clearing and drainage of the soil—in general terms, cultivation—do not materially affect the development and intensity of great floods, is there any respect in which man's work does enhance their destructiveness? This question admits of a definite answer in the affirmative. By obstructing Nature's overflow channels Man draws on himself the chief calamities which floods produce. The bottom lands along streams are just as much a part of Nature's highway for carrying run-off to the sea as the normal channels between banks. They do not come into play so continuously, but they are a part of the natural waterways, and were formerly so used whenever the discharge rose above a certain volume. Man has pre-empted these channels entirely. They contain the most fertile lands, they are easiest to build on, and most convenient for roads and railroads. It is here that population multiplies and wealth accumulates, and whenever Nature asserts her old prerogative of passing her floods down over them, trouble ensues. By cutting off these bottoms, greater work is forced on the ordinary channel, the flood heights are raised, and if once the waters break their bounds, they not only find wealth and people in their way, but are actually far more destructive than in their natural state. Thus Man is, to a large extent, the author of his own misfortunes in the matter of flood destructiveness. The point here emphasized, however, is that Man's interference with natural conditions, which so often leads to disaster, relates to his trespass on overflow channels rather than, as is popularly assumed, on the normal channels of the streams themselves. Contrary to general opinion, Man's operations have, on the whole, improved the natural carrying capacity of streams. There are exceptions, of course, but the rule seems to be as just stated. A striking example is furnished by the recent studies of the Great Miami River, in Ohio. In the four principal towns, where channel changes have been most important, the discharge capacity was found to be from two to eight times as great as in the country above and below, where Man's interference has been slight. Such a condition, of course, cannot be accidental.

The foregoing remarks disclose the reason why flood destructiveness is increasing. It is not because floods themselves are increasing, either in frequency or intensity, but because property subject to destruction is very greatly increasing, and as yet protective work does not keep pace with this increase. In time, this disparity will disappear, or rather be reversed, and flood destructiveness will then show a decrease. Some railroads have already carried protective measures so far that they suffer little, even from the greatest floods.

Reservoirs.—The divergence of view between the Majority and Minority Reports, concerning the conflict of purpose of reservoirs and the impracticability of utilizing the same reservoir simultaneously for flood control and other uses, is probably more apparent than real.

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It would seem that the essential principles are too clear to be capable of dual interpretation. In the first place, the conflict of purpose in reservoirs can be completely overcome by giving the reservoir excess of capacity over storage requirements sufficient to provide for flood control. This, however, is often impossible for lack of site (ignoring, for the present, considerations of cost), and inasmuch as industrial use, municipal supply, etc., must be on a dependable basis, it follows that they must receive first consideration. Subject to this restriction, however, storage reservoirs, as such, will generally be useful in flood prevention. It may happen that a flood-producing storm may come early, when a reservoir is filling, in which case the reservoir might be as effective as if designed for flood control only. Again, the flood storms may arrive late, after the reservoir has become full or nearly so. In that case, the discharge must go over the spillway; but, as some elevation of surface is necessary to produce the spillway overflow, and as the surface of the reservoir, and the storage per unit of rise, are at a maximum at this stage, there will be, even in this extreme case, a certain degree of control.

Nevertheless, the general principle laid down in the report holds. The engineer charged with a project of flood control, in which the use of a storage reservoir is contemplated, cannot take into consideration possible advantages which may or may not accrue. If there is a possibility that they may not accrue, he is bound to assume that they will not. He must make the most unfavorable assumption within reasonable probabilities. Having thus determined what aid can be positively depended on from the reservoir, he must make other provisions for such additional control as is required. It is right here, it seems to the writer, that public supervision of reservoir construction is demanded, and that public aid in the interests of flood control might well be given. For example, if a reservoir site is about to be developed on a stream where the flood problem is important, it might well be that, by increasing the capacity beyond what would be sufficient for storage purposes, the reservoir could also be made to serve the additional purpose of flood prevention. In that case, public aid would seem to be justifiable.

On the whole, the possibilities of combining flood prevention with other uses of reservoirs are sufficient to justify Mr. Knowles' contention that the Committee's report would be strengthened by omitting lines 17 and 18 on page 1225: "Your Committee, however, does not intend to condemn *in toto* the utilization of reservoirs for more than one purpose."

Barriers.—The comment on "Barriers", (*c*) page 1225, seems to the writer most judicious in recognizing only the function of arresting débris. Just now there is current another of those extremist propagandas which have interfered so much with scientific treatment of many of our problems by confusing the public mind with schemes the

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only virtue of which is superficial plausibility. Los Angeles, Cal., is the present victim. A campaign is on there to make barriers the sole, or at least principal, solution of the flood problem in that locality. There has been created a "very wide-spread furore that the only way to control the flood waters [at Los Angeles] is to build myriads of little check dams in the mountains, and, through the detritus caught above each of them acting as a sponge, to soak up all the floods before they leave the mountains."

So far as the writer has observed in reports on this subject, the primary purpose of these check dams is to prevent erosion. Not much stress is laid on their effect in storing water. They soon become filled with detritus, and this, in the general case, will become well soaked with the rains leading up to great flood storms, so that the chances in favor of any material reduction in discharge are altogether uncertain. Immediately following a dry spell, a heavy run-off might be considerably reduced, whereas, with a storm of the same intensity following a series of lesser rains, there might be practically no relief. In this, as in the case of forests, the engineer cannot build on uncertainties.

Outlets.—With some hesitation the writer offers a certain criticism of the conclusion stated in the last paragraph on page 1227. So far as he has followed the official reports on the subject, it is evident that there is an important distinction between the outlets proposed, say, near New Orleans, and natural outlets or accidental crevasses cited as militating against the outlet theory. It is manifest that the action on the regimen of a river of a great crevasse must be different from that of a long overflow weir which comes into play only after the river has reached a certain stage. The writer cannot understand why, if the river safely transports its sediment at that stage, it should fail to do so by preventing any marked rise thereafter through abstraction of the surface flow by a weir. It is very possible that the outlet channel itself might silt up and become ineffective, but that would have nothing to do with the argument just advanced.

Cut-Offs.—Having now reviewed the general features of the report, the writer desires to devote more detailed consideration to the subject of cut-offs. In general, his views coincide with those of Mr. Grunsky. He believes that the ultimate effect of cut-offs is beneficial, providing always that those reasonable and obvious precautions are taken which the circumstances of the individual case clearly indicate. He dissents from the orthodox view, so long held on the Mississippi, that that river has a sort of intelligent purpose in maintaining a particular length of channel, and that it always "seeks" to regain that length if suddenly shortened by the excision of a bend. The truth is that a river in an alluvial soil may perform almost any antics which the physical conditions will permit; but, that such a river has a definite length which

it is always seeking to maintain, is an assumption not substantiated by fact, at least on other streams. Mr. Chittenden.

In illustration of this point, the writer presents a diagram (Fig. 3), showing the changes of a century in about a 50-mile course of the Missouri River. The early cut-off of some of the enormous bends that existed in 1804 has never resulted in any "attempt" on the part of the river, either on this section or above or below, to regain its original length. On the other hand, successive surveys show if anything a contrary tendency; and there is nothing in the long record of navigation on that stream which indicates any ill results to navigation resulting from such cut-offs.*

The writer also questions the correctness of the statement that cut-offs "increase flood heights below them", while "reducing flood heights immediately above". This may be the first effect, but it is not first effects, but permanent results, which must be considered. In the long run, the result, according to the writer's observation, is to draw down the channel above the cut-off until it comes practically to the original grade of the channel below. (It is understood that we are here considering only the case of alluvial streams in comparatively unstable channels.) In the course of time, these effects may extend many miles up stream, and the sharp change of slope at first produced may be distributed so far as to be very slight per unit of distance.

Fig. 4 is reproduced from a photograph of quite an extensive cut-off on the Missouri River which took place under the writer's observation.

The channel shortening, as he now recalls (without verification from the records), was about 7 miles. Much local alarm was felt lest the cut-off should produce a flood below. The writer tried to assure the public that no such thing could possibly happen; that the cut-off in itself would not increase the discharge of the river at all, except as to the small increment from draining the bend, and that there was absolutely no danger. Nothing would allay the fear, however, and there was much talk of taking steps to prevent the cut-off, when certain interests which wanted it to take place rendered judicious assistance (with dynamite, mainly, it was said), and the controversy was promptly settled. At the first gauge below, a rise of a few inches was detected, but no untoward result whatever happened. In this case, the writer feels certain that the final result was a lowering of the old bed above the cut and not a raising of the bed below.

The reports of the Mississippi River Commission abound in references to work designed to prevent cut-offs which might (to cite a

* One naturally questions the correctness of the early maps on which these extreme bends are shown; but, apart from the general accuracy of Lewis and Clark's work, as shown by later surveys, there are authentic references in the journals of other early voyages up the river. It never took less than a day and sometimes two days for a keelboat to get well past Blackbird Hill, opposite the neck of one of these bends, after having arrived in its vicinity.

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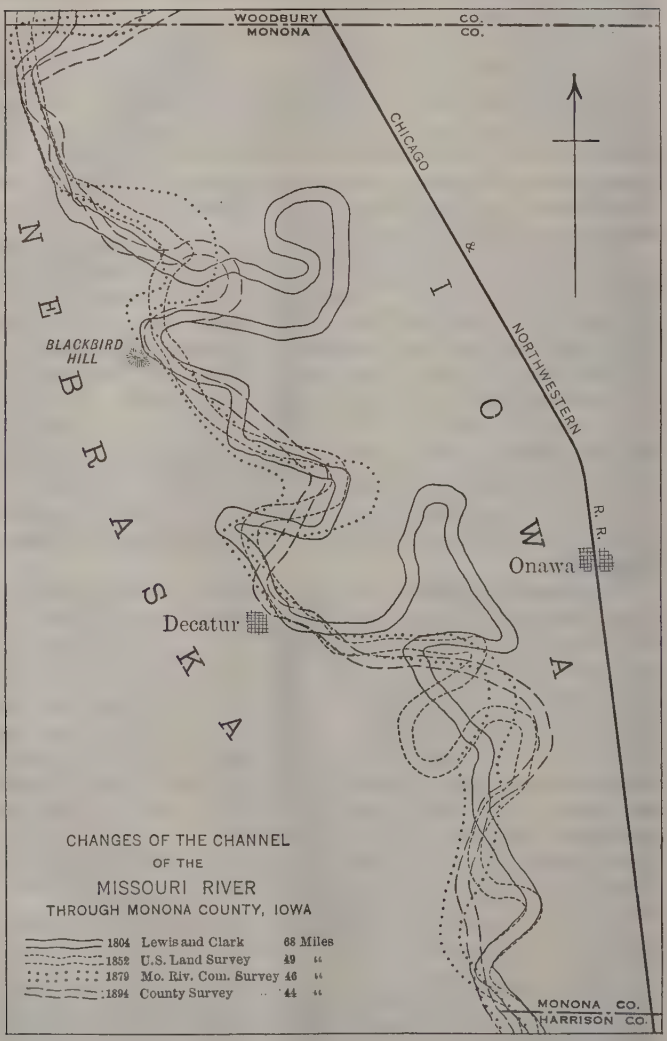

FIG. 3.

FIG. 4.—A NEWLY FORMED CUT-OFF ON THE MISSOURI RIVER. OLD CHANNEL IS TOWARD BACKGROUND ON THE LEFT AND FROM BACKGROUND ON THE RIGHT.

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particular case) "disturb the regimen of the river a long distance above and below, and bring disaster to towns, levees, and other works along the river banks." The results of a cut-off that did take place are traced in considerable detail, showing how one ill effect led to another, until a heavy toll of misfortune had accumulated. All this appeals to the writer, however, as a clear case for the Franklinian philosophy contained in the parable of the horse-shoe nail. If the occurrence of this cut-off had been anticipated by timely protective work above and below, probably nearly all the ill effects which actually developed might have been avoided. The writer would not for a moment criticize official policy in preventing cut-offs on the Mississippi River; he simply criticizes the theory on which that policy is ostensibly based. He believes that the real objection to cut-offs on that stream is not the effect on the regimen of the river (which can be controlled effectively), but on riparian interests. Ports and landings might be ruined and other interests jeopardized in a way that would result in serious hardship; but, aside from these considerations, viewed only as a problem in river hydraulics, the writer believes that a material shortening (say 125 to 150 miles) of the Mississippi River between Cairo and Red River would not, in the long run, injure the regimen, even for purposes of navigation, and it would certainly make the river a better flood carrier and would reduce materially the length of channel to be maintained and of bank to be protected.

The writer would dissent, further, from the distinction which the Committee draws as against alluvial streams in this connection. It is, in many instances, on alluvial streams that this measure can be best applied. The difference between an alluvial stream and one with stable banks is that, in one case, disturbance of regimen may result, and, in the other, it may not, and that, therefore, in using the cut-off on alluvial streams, auxiliary measures are necessary. If these are provided, however, the shortening of the channel should certainly facilitate the natural movement of sediment down stream. Cut-offs have been used extensively on great alluvial rivers, notably on the Danube and other streams of the Hungarian plain.

The cut-off method finds its most effective application in low flat tracts in which stream sinuosity dates from the remote period of emergence of the soil from a status of continuous overflow, and is not a result of present-day processes. Generally, in such locations, the sinuosity is excessive, the slope very slight, and the channel capacity wholly deficient. Extensive cut-offs do not produce excessive slopes and troublesome erosion, but facilitate drainage and carry off flood waters more rapidly. Whether in any particular case a cut-off or series of cut-offs is desirable depends on local circumstances. The most important obstacles are cost and the likelihood of riparian complications. Another consideration relates to the excised portions of

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the channel. If these can be filled up, there is a net gain of land area, for the excised portion is always larger than the new channel. If they cannot be filled, there is a loss of land equal to the new channel. Even in that case, there may be some utility in the excised sections in connection with drainage. By placing flood-gates between them and the new channel, they can be drawn down to the level of low water in the dry season and then be utilized as immense sumps to maintain effective drainage during high-water periods.

Minority Report.—Concerning the several substitutes proposed in the Minority Report, some of them seem to be in the interest of completeness and precision, and some appear to the writer as undesirable. Among the latter are the proposed changes in the forestry discussion. As to (10), on page 1233, it would seem that Mr. Knowles confuses two distinct things. The fact that levees cut off valley storage previously available does not in any way alter the other fact that levees, by raising the flood surface between them, do develop a certain amount of channel storage.

It is important, however, to observe a distinction, which neither the Committee nor Mr. Knowles has brought out, but which justifies to some extent Mr. Knowles' contention that "such storage is ineffective", although for a reason quite different from that which he has assigned. It is entirely true that the volume of water at flood stage stored in stream channels, like the Mississippi and Ohio, is prodigious, including as it does not only storage in the channel proper but the filling of side ravines and tributary valleys and the overflowing of bottom lands back to high ground or levees. The natural impulse is to give such storage great weight in holding down flood heights; but this is clearly inadmissible. Channel storage is mainly absorbed in the preliminary stages of a flood. The case is similar to that of a reservoir which has been filled from the earlier rains and has no space left for the later rains; and though it may still exercise a slight moderating effect on run-off passing through it, such effect is not at all in proportion to the full capacity of the reservoir; and the more prolonged the crest of the flood is, the less will be even this attenuated effect.

The same conclusion may be reached by a hypothetical illustration: Suppose the Ohio River from Pittsburgh to Cairo to be a closed conduit, of cross-section equal to, say, the ordinary low-water section of the open channel; and suppose that, by some mechanical process, sufficient velocity could be imparted to the current in the conduit to carry away all run-off as fast as it should present itself. It is manifest that the discharge at Cairo during rising stages would be greater than with an open channel, due to the fact that no storage had been allowed to accumulate; but this difference would evidently disappear when the maximum stage had been reached and there was no further accumula-

tion of storage. At the critical point of the flood, therefore, the reservoir function of the channel ceases to have effect. Mr. Chittenden.

The writer feels the more freedom in criticizing the Committee's treatment of this matter because he is at the same time criticizing his own previous utterances on the same subject.* The conditions are abstruse and difficult to lay hold of definitely, but the writer believes that they are fairly set forth in the following statement:

Storage in and adjacent to the channel of a stream, which results automatically from an increase of stage, has little if any effect in reducing the maximum stage. It operates to reduce somewhat the rates of discharge during rising stages while storage is actively accumulating, but this effect gradually disappears on approaching the maximum stage where storage accumulation ceases.

Myron L. Fuller,† Esq. (by letter).—The problem which most nearly attains successful solution, whether it be one of business, science, or engineering, is that which takes into account, duly analyzes, and gives proper weight to the greatest number of contributory factors. Mr. Fuller.

Of the factors of flood control or prevention, geology is assuredly one that must be reckoned with. The far-sightedness which is essential to the satisfactory conclusion of any great undertaking demands that every known factor of the problem be considered and the weight given to it that mature investigation shows it has the right to claim.

Nothing is more true than the following statement by Morris Knowles, M. Am. Soc. C. E., in the Minority Report of the Special Committee. "It is unwise to assume that we have at any time obtained all the knowledge possible upon a given subject." Every year sees new advances in our knowledge of engineering problems; and our ideas as to the precautions necessary in engineering undertakings are continually changing. Factors hitherto unknown or neglected are almost daily found to be of importance.

Most of the great failures in engineering works, as well as in other lines of business, have been due directly or indirectly to overlooking some inconspicuous factor or neglecting some element of apparent insignificance. For example, the lack of knowledge as to the weakness and slipping planes in shales, or the failure to realize their significance, has resulted in the failure of several large dams and reservoirs within the past few years, and the inadvertent omission of some engineering factor has played an important part in the collapse of bridges and other steel structures.

The writer does not maintain for a moment that the success or failure of flood prevention or control measures is dependent on geological investigations, but he does contend that geology is one of

* "Flood Control, with Particular Reference to Conditions in the United States", presented to the International Engineering Congress, San Francisco, September, 1915, pp. 16 and 22.

† Boston, Mass.

Mr. Fuller. the factors—sometimes one of marked importance—which should be considered in any comprehensive investigation looking to the prevention or control of floods.

Insufficiency of Precipitation Data.

Notwithstanding the great efficiency of the Weather Bureau, rainfall data, chiefly because of the comparatively short periods during which records have been kept, are admittedly insufficient for the determination of the maximum possible or even probable precipitation in a given area.

It is true that a certain approximation to the probable maximum precipitation may be made from an inspection of the rainfall records throughout the general province or climatological zone of which a given district is a part, by applying the maximum precipitation recorded elsewhere to the area under consideration. As is well said in the Report of the Special Committee (page 1220), however, "it is improbable that the maximum precipitation that may occur at a given locality has yet been recorded."

Taking the maximum rainfall recorded for the province or zone, and the maximum known accumulation of snow for the same latitude and altitude within the zone, it is possible to estimate the probable height of a flood under existing conditions of channel. The height calculated, however, is by no means free from likelihood of error, and it is entirely possible for the water to rise many feet higher than the estimated limits.

Geological Clues to Flood Heights.

Flood-Plains and Terraces.—In the absence of sufficient rainfall data, it is fortunate that geology furnishes, in many instances, a clue to the probable heights of flood-waters. The clue referred to is that afforded by the flood-plains and terraces, which often give evidences of heights of water greater than those which would be indicated by the computations from greatest recorded rainfall.

Generally speaking, there is little evidence that floods are either more or less pronounced and frequent at the present time than in the immediate past. This, of course, does not apply to the more remote geological past, when climatological conditions were radically different.

The evidence afforded by flood-plains is not altogether simple or universally present. Mountain streams, having steep gradients, are usually cutting rather than depositing, and flood-plains and terraces are commonly absent, although even in mountainous regions elevated pockets of sand often afford clues to the high-water limits. In flatter regions, such as the greater portions of the Mississippi and Ohio Basins, the streams are generally aggrading or filling their

valleys, at least locally, and are commonly bordered by flood-plains or terraces. Mr. Fuller.

The flood-plains are fundamentally constructional in origin, being built up of the relatively fine deposits laid down when the waters of the streams overflow the normal channels and spread out with checked velocity. The upbuilding continues, unless new geological or artificial factors intervene, until the surface reaches nearly the level of the highest floods. During the later stages, the upbuilding is very slow, and the floods overtop the flood-plains only at long intervals, perhaps of centuries. In the majority of localities the flood-plains have not reached their final stages, especially where, because of constrictions of the channels or other causes, the currents are unusually swift, but are still overflowed to a depth of several feet by the greater floods.

The uncompleted stages of the flood-plains introduce large elements of uncertainty, and the problem is still further complicated by the fact that portions of the once completed flood-plains may have been subsequently eroded away, leaving surfaces much lower than the levels reached by the floods. In other words, a series of terraces is left, only the uppermost members of which are of significance in determining flood heights.

If this was all, the problem would still be comparatively simple, but further difficulties are introduced by the fact that, in some instances, the whole river valley has been cut in glacial or other sands and gravels of a past geological period. In this down-cutting, terraces similar in appearance to those of the flood-plains are commonly left at various levels.

The determination of which of the general series of terraces in a given valley are due to upbuilding, and are, therefore, of significance in the determination of probable flood levels, and which are due to down-cutting, and are without present significance, is often difficult, and the discrimination can seldom be left to the engineer with safety, or even to the academic geologist of the average college faculty. The discrimination can be made, but it requires a practical and experienced field geologist to make it with certainty.

The flood-plains and terraces, notwithstanding their complexities, afford a line of evidence that should not be neglected in any comprehensive investigation of flood control. When the writer was engaged in Ohio on Government geological surveys, ten years ago, the question was several times asked in regard to towns or their water supplies: "Is there any danger from floods?" The answer was that the towns were situated on flood-plains built up by repeated overflows of the streams, and that similar overflows were bound to occur in the future unless artificially prevented. The writer did not necessarily expect

Mr. Fuller. a flood of the proportions of that of 1913 so soon, but it was bound to come eventually.

Geological Obstructions to Streams.

Geological obstructions to streams, because of their obscurity, are often overlooked—or their importance under-estimated—in flood-control studies. Two distinct types may be mentioned: the boulder delta and the rock barrier.

Boulder Delta.—Existing deltas or fans of boulders or coarse gravel are manifest obstructions, and are recognized by engineers as such. They occur chiefly where side streams having steep gradients issue from hills or mountains into broader valleys where the streams are more sluggish and are usually engaged in building up their flood-plains. Often, the main streams are pushed aside and forced close against the opposite banks, where they flow in comparatively narrow and deep channels. They are obstructions which have an unmistakable and calculable effect on flood height.

The deltas which are not pre-existing, on the other hand, introduce important but unknown factors. A severe rain over the water-shed of a small tributary is felt almost at once, and the stream commonly responds long before the water rises very materially in the main valley. The result is that enormous volumes of material are often swept into a river, causing great accumulations of gravel or boulders where no deposits may have previously existed. It is true that, as the main river rises, many of these accumulations may be swept away, but, in the meanwhile, they are a very real obstruction, with an important influence on flood heights. Their final destruction may inaugurate flood crests endangering the whole country below. Under other conditions, the deposits may become permanent features, at least for a considerable period of years.

The boulder or gravel deltas of this type are certainly worthy of attention in flood-control studies, and geological investigations of their possible effects are highly desirable.

Rock Barriers.—In many of the valleys in the Ohio and Mississippi Basins there are buried rock channels much deeper than the general rock floors over which the streams now flow. The latter commonly mark the depth to which the scour of the river extends during flood periods at the present time; the deeper channels mark the former positions of the river under different conditions of level or discharge. Sometimes, the present channels coincide with the older rock gorges, in which case the scour may extend to somewhat greater depths. Sooner or later, however, the streams have to return to the general rock floors, usually accompanied by sudden rises in the bottoms. This has an important effect in checking flow and in promoting deposition.

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Although the importance of the rock barrier is less than that of the delta, its influence is sufficient to warrant taking it into account in the flood-control problem.

Geological Factors of Run-Off.

The principal geological factors of run-off are: the grain of the soil, the thickness of the soil, and the nature of the underlying rock, including composition and fissuring.

Grain of Soils.—The grain of soils, using the term "grain" in its broader sense to include the arrangement and packing of the particles as well as their size, is of the greatest importance in run-off and flood studies of a given water-shed. A proper study of the grain includes both laboratory tests and field examinations. The former consist of sizing tests or analyses by passing the material through a series of standard screens of various meshes, and from them may be computed the rate of percolation under different conditions. Direct tests on blocks of soil cut from the ground, where the material is sufficiently firm to hold together, have been found, in the writer's experience, to afford still more dependable results. All measurements or estimates of rates of percolation apply, naturally, only to unfrozen soils, hence are of value only in seasons where the ground is free from frost.

Classification of Soils.—Accompanying the percolation estimates and tests mentioned—which are as much of an engineering as of a geological nature—field examinations must be made, and the soils classified, mapped, and their areas determined. These can be made best by a geologist or soil expert. Samples, at the best, can be taken at only a limited number of points, and must be supplemented by a soil classification of the entire area, if the work is to have any real value. The geologist's knowledge of the general character of residual soils, or those resulting from the weathering of different types of rock, of the nature of the various classes of glacial deposits, and of the composition of the alluvial accumulations, is of the greatest assistance in this branch of subsidiary investigations in flood-control studies.

Depth of Soil.—The depth of soil is of much greater significance than is commonly realized. The writer's ground-water studies in some thirty States have shown that rocks free from soil coverings seldom contain much water, and that the same rocks, when covered by a thick soil mantle, afford abundant supplies. Conversely, when the soil is thin, the immediate run-off is much greater than where it is thick. Therefore, geological studies to determine the depth of soil are of considerable importance in run-off studies.

Composition of Rock.—The absorptive capacity of the underlying rock is a factor of significance in flood control. The porosity or absorptive capacity varies greatly in rocks of different classes, ranging

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from ½% or less in quartzites to 5% in shales and limestones, and 25% or more in sandstones. Many engineers can recognize the various major varieties of rock, but the numerous errors in determination which are indicated by the reading of engineering literature show that it would be far safer to rely on a geologist than to leave rock determinations to guesswork.

Fissuring of Rocks.—Fissures in rocks have been mentioned by several engineers as a factor in natural flood regulation. Broadly speaking, rock fissures are of four general types: cleavage planes, bedding planes, joints, faults, or other fracture planes, and solution passages.

The cleavage planes, limited chiefly to slates, and the bedding planes of sedimentary rocks, are commonly potential rather than actual fissures, and, even when open sufficiently to admit water, take it up and deliver it very slowly. They affect the flood heights but little, their office being that of feeders and regulators of low-water flows.

The openings along joint or fault planes are rarely more than a fraction of an inch in diameter, and although of more importance than cleavage and bedding planes, they are likewise regulators of ground-water flows in low-water stages, rather than factors in floods.

The solution passages in limestones, on the other hand, are often many feet in diameter. The Mammoth Cave of Kentucky, with its large passages several miles in length, is an example of this type. It is now abandoned by the stream that formed it, but, elsewhere, similar channels are still occupied by underground rivers. The writer has seen such a stream on the outskirts of the Ozarks, in Missouri, where a torrent several feet in width and a foot or more in depth rushed along underground with rapids and falls like those of a surface stream.

Such solution passages are natural tunnels capable of carrying immense volumes of water. Nevertheless, notwithstanding their frequency and large capacity, they are rarely elements of importance in floods, as they replace rather than supplement the normal surface drainage. Practically all of them are above the normal drainage levels of the main streams, or those in which floods are likely to occur, and lead from limestone sink-holes or other openings in the hills down to the main valleys which determine the levels of their outlets. Although they can lead water down to a flooded stream, they can seldom, in the absence of lower outlets, take it away.

If clogged, a certain quantity of water is sometimes held back, but this is often compensated by the breaking loose of obstructions previously formed in other passages, and the setting free of waters to be added to the flood.

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It is to be noted, however, that in a few localities streams of considerable size are known to flow at heights of several hundred feet above adjacent streams. If in a limestone region, a solution passage might take away a large proportion, if not the entire volume, of the higher stream, but the quantity taken from the upper stream would be added to that of the lower, where the flood conditions might be even more serious.

It is apparent, therefore, that solution passages may be a real factor in floods, and, though not a common element, are sufficiently frequent to demand recognition. In storage reservoirs, where the water is held above both the normal drainage and ground-water levels, they are often of paramount importance, and great precautions are necessary to prevent damage or loss through their agency. The reservoir failure at Johnson City, Tenn., on January 10th, 1913, is an example of failure from neglect of solution passages.

Geological Factors of Ground-Storage.

The storage capacity of the sands and gravels of alluvial deposits adjoining a stream is a factor of considerable importance in flood regulation. The pore spaces of such materials commonly range from 25 to 35% of their volume, and although the residual moisture already present may reduce this very materially, there is usually an available porosity of from 10 to 25 per cent. In other words, 10 ft. of gravel will take up the equivalent of from 1 to 2½ ft. of water. Of course, if the rise of the flood is slow, the gravels will often fill with ground-waters from the hills, but, in sudden floods, the ground-waters have insufficient time to respond to change of level, and adjacent sands and gravels will become filled from the river. The inflow into the gravels is often quite rapid. In one case observed by the writer a cellar 100 ft. from a stream subject to sudden fluctuations was filled to stream level with a lag of less than 2 hours.

It is apparent, therefore, that, under favorable conditions, the storage capacity of adjacent deposits may be a prominent factor in floods, and geological studies, to determine their location, extent, and availability, are highly desirable. In such studies, the aim should be to determine, in addition, the presence or absence of buried rock masses which might limit the storage capacity of a particular formation. Buried barriers of rock opposing the free movement of the underflow and bringing it to the surface to be added to the floods should likewise be located whenever possible. The position of buried channels, if present, should be determined, for they may provide outlets for the underflow at high-water stages when not available at low stages, or may at least afford additional storage.

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Limited Value of Sedimentation Basins.

Moving water will always carry its full load of sediment if the material is available. It is a geological principle that, if it does not already have such a load, a stream will pick it up from its bed or banks until it is fully loaded. The greater portion of the coarse material under transportation by a stream is carried only a short distance. It is picked up where the current is accelerated, the scour often extending to depths of 30 ft. or more, but it is deposited in large measure as soon as the current is checked. Only the finer and relatively harmless silts become a permanent addition to the water. In the course of time, by innumerable repetitions of the process described, the coarser material is carried to points far distant from its source, but very little except the finer silts is transported to any considerable distance during a single flood.

For these reasons, it appears that, as a general rule, sedimentation basins may be expected to be of limited value, although, as local protection, they may occasionally be entirely successful.

Levees.

A levee system, by confining the current of a stream, tends to intensify the scour, and, for the same reason, subsequent deposition may be hindered. In the long run, river bank erosion is reduced, and such deposition as occurs is probably less harmful than along an uncontrolled channel.

Outlets.

The value of natural or artificial outlets depends to a considerable extent on geological conditions. An outlet may present a shorter line of discharge and a straighter course than the existing channel; but, unless the water can get into it freely, it is not likely to be utilized by a stream. The inlet to it must be free from obstructions and in line with the natural swing of the current, and the banks must not cave (in proportion to the volume of water) more than those along the existing outlet.

If any considerable volume of water follows a new outlet, temporary relief is, indeed, afforded, but the improvement is not permanent. Either the new or the old channel will soon silt up, and the final condition will be no better than before. The Hwang-Ho, in East China, shifted its mouth a distance of 275 miles in 1852, but flood conditions are said to have been accentuated rather than relieved.

Necessity for More Comprehensive Studies of Streams.

The Government, in connection with its comprehensive and detailed studies of the Mississippi and its larger tributaries, has secured very

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complete data on erosion, transportation, and deposition by low-gradient streams, especially in their lower reaches. There is, however, a deficiency of data pertaining to the upper reaches and at the headwaters.

It is evident that the conditions in the upper portions of most large streams are entirely different from those in the lower portions. In the latter, notwithstanding the wholesale caving of the banks, the excavation of cut-offs, etc., deposition predominates, and stream beds, flood-plains, and estuarine deposits are being gradually built up. On the middle reaches erosion and deposition are more nearly balanced, and the conditions as regards flood-control are essentially different. In the upper reaches and head-waters, erosion usually predominates, and flood conditions differ materially from those of either the lower or middle reaches.

In many of the smaller drainage basins, such as those of several of the Ohio tributaries, all three conditions are represented within comparatively short distances, and no one set of laws of erosion and deposition will apply. The data derived from the Mississippi studies may be applicable to the lower reaches of these streams, but there is a decided deficiency of information as to the behavior and work of the streams in their upper sections.

The result of this deficiency of knowledge of the behavior of the swifter streams is illustrated by the destruction of many railroads at various times in the past in the canyons of the West, especially along the San Pedro line between Los Angeles and Salt Lake City, which was rebuilt for long distances after its destruction by flood about 6 years ago.

The question of run-off needs more careful consideration, especially the effect on it of forestation, with a view of fixing quantitative values that may be depended on with more certainty than is now possible.

Flood depths and flood velocities in mountainous and hilly regions need more extended investigation in order that the limits of danger may be more definitely fixed. This is often even more important than similar investigations in the flood-plain sections, as the floods are more sudden, more uncontrollable, and more destructive than in regions of the latter type.

The size and nature of the material available for transportation are also important factors in floods. Structures that might withstand the water alone may fail under a bombardment of boulders, often hundreds and sometimes thousands of pounds in weight, which are swept along by mountain torrents. More information is also needed as to the effects of bends of various radii in the canyons and valleys, as well as the influence on floods of narrowings of the valley walls.

Mr. Fuller. Many of the factors mentioned are geological in their nature, and their investigation would naturally take place in connection with that of the numerous other geological problems involved in flood regulation or control.

Final Statement.

A few of the geological factors entering into the problem of flood control and prevention have been stated briefly. Many others would undoubtedly develop in the field. The writer believes that some of these factors will be generally acknowledged as of unquestionable importance. At any rate, it is not safe to pass them by without mature consideration in any broad investigation of floods.

Mr. Matthes. Gerard H. Matthes,* M. Am. Soc. C. E. (by letter).—The writer endorses very heartily the recommendation contained in the Report of the Special Committee on Floods and Flood Prevention, urging the establishment and unification of systematic rainfall, run-off, and flood observations covering the entire United States in far greater detail than has yet been attempted; also, the recommendation contained in the Minority Report, urging the creation of a special agency, supported by adequate appropriations, for carrying on this work. Our knowledge of floods in general is sorely in need of systematic development; the recommendations referred to indicate the first logical step to that end.

There is a crying need for carefully compiled chronological flood data showing dates of occurrence, flood heights, distribution and quantities of rainfall, rates of run-off, and notes as to what tributaries were directly responsible for the formation of flood crests, for all rivers of importance, and including also the lesser streams which, owing to geographical or commercial conditions, present special flood problems. The writer believes that the Committee can render the Profession no greater service than by outlining a plan of procedure for collecting, collating, and publishing such facts systematically.

A review of available river stage, stream flow, and rainfall data reveals a deplorable lack of accurate statistics pertaining to either floods or to extreme low-water conditions. Yet, these two subjects are of more economic importance than any others pertaining to the regimen of streams. Floods are at present attracting unusual interest; low-water stages and droughts will probably become of more and more interest as time goes on. As matters now stand, the practising engineer, who is required to study and report on means for abating flood damage in a given locality, finds three distinct classes of information at his command:

1.—River stage, stream flow, and rainfall observations, made under the direction of Federal and State bureaus, and of certain

* Dayton, Ohio.

corporations, all of which observations are more or less readily obtained in printed form. In this class belong also miscellaneous data published in technical journals. Mr. Matthes.

2.—Data pertaining to floods which antedate the periods covered by existing records, and concerning which but little has been published in technical publications.

3.—High-water marks.

Each class of data possesses limitations of its own, as will appear from the following considerations. Data of Class 1 are generally regarded as affording the most reliable information. Unfortunately, river stage records are available for comparatively few streams, and rainfall records are obtained at illogically distributed points. The average river stage record does not cover more than 20 years, a period entirely inadequate for studying floods with reference to their frequency. Even 40-year records, of which a number are in existence, are inconclusive in this respect, because they contain rarely more than one extraordinary flood. The principal short-coming of such records, however, is that they were obtained primarily to supply information of a general nature, and not to afford specific information regarding floods. This fault has been recognized, and is being remedied. The United States Geological Survey and the United States Weather Bureau, in recent years, have instructed their observers to record the maximum or crest stage of each flood, and in case of flood stages caused by back-water from ice jams, to make notes to this effect. Previous to this there was no definite practice, some observers reporting readings taken at the customary hour, others reporting the crest stage without stating in all instances the time of its passing. Others took sufficient interest to obtain a series of readings during a flood, but the Bureau, in publishing the record, contented itself with averaging them and printing the meaningless figure thus obtained as representing the average gauge height for the day. This practice is still in vogue in some quarters. Many flood heights published by the Signal Service and the Weather Bureau in their earlier reports, on investigation, have proved to be distortions caused by ice jams, and are not to be taken as indications of flood discharge. Failure on the part of engineers to investigate and check up such records before using them has resulted in the indiscriminate use of observations which are not comparable. Some engineers believe that inaccuracies and inconsistencies of the kind referred to are not likely to affect their conclusions one way or another. Others, unaware of the defects, have taken the records for gospel truth, and with painstaking care have made them the basis of mathematical and graphical studies, assigning to them a value which is wholly unwarranted.

Mr. Matthes.

There is an urgent need for a complete revision of all published river stage records and stream flow data, in so far as they relate to floods. The importance of this matter cannot be over-emphasized. A revision of this kind was undertaken some time ago by the Water Supply Commission of Pennsylvania for the streams of that State, as a result of which many of the theretofore published maximum rates of run-off have been materially increased. The writer's studies in this field lead him to believe that probably 50% of the maximum run-off figures published by various authorities, among them the much quoted ones of the late Emil Kuichling, M. Am. Soc. C. E., do not represent crest stages at all, but are based on unscientific observations of the kind just alluded to, and are not even reliable 24-hour averages. The seriousness of this state of affairs is not generally recognized, and deserves the earnest attention of the Committee.

It is generally recognized that the number of river and rainfall stations should be greatly increased. Co-operation between the Federal Government and individual States has given good results in some cases. As an instance of what may be accomplished by a State acting independently, may be cited Pennsylvania, which in 1907 took over from the U. S. Geological Survey a score of river stations, and has increased this number until at present observations are received from more than 100 stations.

The development of our knowledge of floods by continuing existing records and starting new ones is only one phase of the subject; the other phase is to gain knowledge concerning floods which took place before regular observations were begun. This involves the collecting and collating of data of Class 2.

Much valuable information concerning floods, their heights and causes, is obtainable by consulting the files of historical societies, public and private libraries, newspapers, old diaries, private records, and unpublished matter of various kinds. Extensive researches by the writer in this field have taught him that there is a vast quantity of good material awaiting him who will take the trouble to unearth it. Such material, when properly interpreted, and utilized with care, so as to eliminate erroneous data, can be made of great value, covering as it usually does the major portion of the history of any river, in point of time. Obviously, a 150-year record, consisting of fairly complete data relating to great floods, will be of much greater value in determining a future flood control policy, than a 40-year record of daily gauge heights obtained on the same stream by a $5 per month uneducated observer.

Some attempts have been made to compile flood records for a few of the more important streams of the United States, the data extending back to the earliest days of settlement. The reason more has not been

done along this line is that it has not been made anybody's business to attend to it, and because, also, of a certain amount of prejudice generally prevailing against the use of non-technical data. Federal and State bureaus have made little effort to collect information of this kind, principally for lack of appropriations, and partly because of inertia to be overcome in stepping out of the beaten paths of regular routine. The nearest approach to work of this kind was undertaken by the Water Resources Branch of the U. S. Geological Survey, when, in Water Supply Papers Nos. 96, 147, and 162, it published accounts of destructive floods during the years, 1903, 1904, and 1905, respectively, including many references to early floods. The Weather Bureau has also listed early floods for a few streams. Without doubt, both these bureaus possess a large quantity of material, which, if carefully checked and amplified, would become most valuable contributions to flood literature. In its present condition, such information lies dormant, and is of no benefit to the Profession. Mr. Matthes.

Reverting next to Class 3, High-water marks, it appears that mankind, since the earliest times, has manifested a keen interest in the destructive action of rivers. Before the days of regular observations, it had become a frequent practice to perpetuate by permanent marks the height attained by great floods. The extent to which this has been done along the streams in the early settled portions of the United States is remarkable, as will be testified to by those who have had occasion to make systematic search for such marks. Here, as elsewhere, in dealing with information pertaining to floods, it is of the utmost importance to guard against errors, and it is necessary to check the marks, not only against each other, but by comparing the information which they furnish with that obtained from accounts and other data. When made on factories, mills, pumping stations, and bridges by the mechanics, millers, or engineers thereof, such marks, as a rule, are well recorded and reliable.

The present condition of our knowledge of floods is such that few practising engineers are placed in a position where they can utilize to advantage any one of the three classes of flood data here discussed. Lack of facilities, and lack of access to old files or other sources of information stand in the way, and frequently the problem must be solved by guesswork where reliable data might have been used had they been available in conveniently accessible form. It should be made the province of Federal or State bureaus to remedy this condition. To do so calls for considerable research and academic work, neither of which should devolve on the practising engineer, or become a source of expense to his client.

In conclusion, the writer wishes to state that it is to be regretted that the duties of the Committee were limited to the investigation of flood matters only; they should have included an investigation of low-

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water conditions. The subjects are closely related, and the methods which may be utilized for systematizing the knowledge concerning the one could without doubt be made to apply to the other. It is to be hoped that this important feature will not be overlooked when an agency is created such as suggested in the Minority Report. Such an agency could handle advantageously matters relating to low as well as to high water. Great floods have left their imprints where they may be seen for generations to come; but low-water stages have left no such records behind them. Unless an effort is made soon to trace back the happenings of this class which took place in years past, it will become increasingly difficult to glean much on this important subject.

Mr. Barrows.

H. K. Barrows,* M. Am. Soc. C. E. (by letter).—The writer has been much interested in the report of the Special Committee on Floods and Flood Prevention, the Minority Report by Mr. Knowles submitted therewith, and the subsequent discussions by Messrs. Eakin, Hill, Leighton, Grant, and Groat.

The writer is in general accord with the suggestions made by Mr. Knowles in the way of amplifying the report of the Committee, and particularly with regard to a special National agency for making general studies of the subject of floods and flood prevention and procuring systematic and comprehensive data necessary for carrying on such studies.

Mr. Grant states the situation exactly when he says that data are not now available with which to design intelligently and economically the works to carry out the purposes contemplated by the enormous appropriations for river regulation and water conservation which have been urged continually before Congress.

The Water Resources Branch of the United States Geological Survey is the only Government Bureau carrying on systematic river measurements, and, up to the present time, its appropriation for this purpose has been very limited. The entire amount appropriated annually for gauging purposes is only $150 000, which is only partly available for river measurements and must also be used for the administrative and other expenses of this Bureau. This sum is absurdly small, and, in the writer's judgment, the Committee could well make a specific recommendation to aid in the advancement of our knowledge relative to floods and other river conditions by urging strongly upon Congress that the amount of this appropriation be increased very materially. When the great value of such data is considered from the point of view, not only of flood investigation, but of water power and water supply in general, it would seem that the annual appropriation for this purpose should be at least $500 000.

The discussion of the use of levees in flood prevention brings out clearly the fact that on large interstate streams a comprehensive

* Boston, Mass.

scheme for the entire river should be developed, as the use of levees often means a change in hydraulic gradient at flood times extending many miles up stream from the point of the works. Mr. Barrows.

Although the necessity for the use of levees in the lower Mississippi seems to have been proved, the writer is not greatly impressed by the figure of 1 365 000 000 000 cu. ft. cited by the Committee as the storage capacity of levees on the Mississippi between the mouths of the Ohio and Red Rivers. Although the figures in themselves are large, it must be kept in mind that the effect of storage capacity on a stream depends on the tributary drainage area as well as the quantity of storage. The total drainage area of the Mississippi above the Red River is about 1 259 000 sq. miles, and the 1 365 000 000 000 cu. ft. of storage capacity between levees corresponds to a little less than ½ in. in depth over this drainage area, a quantity which obviously of itself is not of great importance in retarding flood waters. Considering only the drainage area of about 329 000 sq. miles between the Ohio and Red Rivers, the storage in the levees would be only about 1.8 in. in depth over this area, or not enough to affect greatly the run-off of even this smaller district. The writer agrees with Mr. Knowles in this respect, that the potential storage over the surrounding country and overflowed land is much greater than the volume confined between levees. The important function of the levees is in increasing the carrying capacity of the channel and confining the flow, rather than in storage.

The recent report of the Miami Conservancy District is an excellent example of an intelligent solution of the flood problem on one stream wholly within one State. For the larger or more extended problems involving interstate streams, obviously, studies should be made by some National agency, and the writer believes that the Committee might well take a more positive stand on this matter and urge the creation of such an agency, supported by an adequate appropriation.

N. C. Grover,* M. Am. Soc. C. E. (by letter).—With the occupancy of the river banks and bottom lands by cities, towns, and farms, the damages caused by floods have increased, until the problem of their control has become of National importance. Recent disastrous floods have led to the appointment of municipal, State, and Federal commissions for the study of local conditions or special phases of the flood problem. The appointment of a Special Committee of the American Society of Civil Engineers for study and report on floods and their prevention is a manifestation of the interest in and importance of the problem. Mr. Grover.

The duty of the Society to make a broad, comprehensive, and unbiased study of the flood situation having been recognized by the

* Washington, D. C.

Mr. Grover.

appointment of this Committee, a heavy burden of responsibility rests on the membership in its attempt to guide the various governmental organizations to an adequate solution of the flood problems. The Society must now see to it that this responsibility is discharged properly and adequately, and its final action should be sound in principle, broad in scope, and definite in application, so that it may serve as a basis for safe, adequate, and unbiased legislation.

At the present time flood work is scattered through many organizations. Several States and municipalities have investigated local flood problems. The United States Weather Bureau has developed an efficient service for flood warnings on the principal rivers of the country, and has collected many valuable records of river stage. The United States Geological Survey has collected many records of flood discharge, as a part of its work in systematic stream gauging, and has made topographic maps of river basins and of reservoir sites that are invaluable in a study of floods and their control. The Corps of Engineers, United States Army, has charge of all construction on navigable streams, and, incidentally, has collected much information relative to floods on such streams. Each of these various organizations has conducted its work in full recognition of the work of the others, but with little actual co-operation. There has been, however, no organization to undertake a broad, comprehensive study of the whole problem.

Flood problems range in complexity and importance from those surrounding a local flood affecting only a small area in a single State to those involving the welfare of the people of several States, or even important international questions. Present methods of treatment are almost invariably local and piecemeal, without proper consideration of the general situation, of possible antagonistic results of different projects and methods, or of the possibility of disastrous effects of projects on unimproved parts of the river system. They do not bring about, therefore, that general orderly improvement in the condition of river stage and erosion that should result from correct and adequate methods of treatment. The co-ordination of projects according to sound and harmonious methods should also eliminate useless efforts and waste, and secure the desired results at a minimum cost.

The elements of a general programme for flood control, on which recommendations should be made in the report of the Society, are:

1.—The development by appropriate research of a fuller knowledge of the laws of river hydraulics and physiographic processes which is prerequisite to a sound practice of river control;

2.—The expansion and improvement of methods of control, on the basis of fuller scientific knowledge, including, possibly, vertical as well as horizontal control of streams, with con-

sequent reduction of necessary levee heights, contraction of levee systems, and elimination of harmful reactions between different projects and methods;

3.—The co-ordination and standardization of the collection of data as to quantity of water, débris in transit, and consequent adjustment of grades and channel forms;

4.—Surveys necessary as a basis for the design of regulatory works, estimates of cost, and of the effects of such works;

5.—Agencies to be used in collecting data and in building and operating the necessary structures;

6.—Division of cost of such structures among the organizations co-operating, or the parties benefited;

7.—Possible combination of flood control, navigation, water power, drainage, and irrigation, under the same general regulative programme;

8.—The organization of a special Federal bureau equipped to execute a full programme of river improvement.

The Report of the majority of the Special Committee on Floods and Flood Prevention, which was presented at the Annual Meeting, was discussed by the minority as a Progress Report. The Chairman, later, requested that the report be considered as the Final Report of the Committee. It is unsatisfactory to the writer, however, either as a progress or a final report, largely because of its apparent lack of clear insight into several important phases of the flood question and of definite suggestions or recommendations. It calls attention only to the need for additional and standardized physical data, and discusses certain suggested methods of flood control in a manner that favors those now in vogue. It ignores the necessity for developing a science of river hydraulics and improving the practice of river control, for active co-operation of agencies in collecting data, in building and operating structures, and in dividing costs, and for the co-ordination of all flood work under one directing head and in one programme. The Minority Report is a great improvement on that of the majority, in many respects, but does not accomplish the purposes, as they appear to the writer, of a final report of a committee of this Society.

It is believed, therefore, that both the Majority and Minority Reports should be received, the Committee discharged, and a new committee appointed to consider again the important problems involved, in the hope that a more comprehensive and definite report may serve as a basis for the final action of this Society on the important problems involved in the flood situation.

Mr. La Rue.

E. C. La Rue,* Assoc. M. Am. Soc. C. E. (by letter).—The Committee has wisely called attention to the paucity of data available for studies of flood control, but it does not seem consistent in the same report to draw definite conclusions with respect to the relative merits of the various methods of flood control and flood prevention. The writer agrees with the statement of Mr. Knowles, on page 1230, that the Committee has given a single view "on controversial subjects on which it must be recognized that much more information can and should be obtained".

For example, on page 1228, the Committee says:

"As you proceed down stream the influence of reservoirs on flood prevention rapidly diminishes, and the influence of levees correspondingly increases in importance as a method of flood protection. On the lower alluvial reaches of long rivers, such as the Mississippi and Colorado, they afford the only sure means of flood control."

This statement is not correct when applied to the Colorado River. The writer has recently completed a report, entitled, "The Colorado River and its Utilization",† in which it is shown that reservoirs may prove an effective means of preventing floods on the lower Colorado. The area of the Colorado River Basin is 244 000 sq. miles. The precipitation and run-off from the lower half of the basin is small compared with that on the upper half. At Yuma, Ariz., 92% of the annual run-off is contributed by that part of the drainage basin lying above the Utah-Arizona line, which is 700 miles above Yuma. Green and Grand Rivers, which unite to form the Colorado in southeastern Utah, drain 70 300 sq. miles, which is only 28.8% of the Colorado River Basin; yet these rivers contribute 76% of the water that passes Yuma. A dam constructed to raise the water level 270 ft., immediately below the junction of Green and Grand Rivers, would create a reservoir having a storage capacity of 8 600 000 acre-ft. On June 14th, 1914, a maximum flood of 137 000 sec-ft. occurred at Yuma. The crest of this flood at the junction of Green and Grand Rivers was 120 000 sec-ft., and passed that point on June 3d. By utilizing the reservoir site at the junction of the Green and Grand, this flood of 137 000 sec-ft. at Yuma could have been reduced to 17 000 sec-ft. This reservoir site is 880 miles by river above Yuma, and about 1 000 miles above the mouth of the Colorado. It is clear, therefore, that in this basin, at least, the influence of reservoirs on flood prevention does not rapidly diminish down stream. The Colorado tends to overflow its banks in the vicinity of Yuma at the 25-ft. stage on the gauge. The average carrying capacity of the channel at the 25-ft. stage is about 50 000 sec-ft. The writer, therefore, believes that nearly all

* Salt Lake City, Utah.

† U. S. Geol. Survey, Water-Supply Paper 395.

overflow can be prevented on the lower Colorado, and that the flow can be regulated to meet the demand for water for irrigation, if the reservoir site at the junction of Green and Grand Rivers is utilized, and detention basins and reservoirs are constructed on the San Juan and Gila Rivers for the purpose of reducing the violent floods that happen occasionally on these tributaries. On the lower reaches of the Colorado the river bed is being slowly built up, and at the present time the river is flowing on the highest ridge of its delta. When the flow of the Colorado is regulated, the cost of constructing levees and bank revetments will be reduced to a minimum. It is probable, therefore, that the final plan for the control and prevention of floods on the Colorado will involve the construction of reservoirs, detention basins, and levees. Mr. La Rue.

As to the proper methods of controlling and preventing floods, there are nearly as many opinions as there are engineers engaged in the study of the problem. Each drainage basin or river system presents a different problem, and perhaps this fact accounts for the divergence in the opinions expressed by engineers as to the effectiveness of the respective methods of flood control and flood prevention. The opinion of each engineer will probably be influenced largely by conditions in drainage basins with which he is familiar.

The Committee, no doubt, did not intend its report to be interpreted as an invitation to engineers to discuss the relative merits of reservoirs *versus* levees as a means of flood control, but the discussions that have been published indicate a tendency in this direction. To the writer it seems no more consistent for engineers to discuss the relative merits of the various methods of flood control without confining their discussions to conditions in a particular basin than it would be for two engineers to debate the applicability of the Cippoletti weir *versus* the current meter as a means of measuring water, without agreement as to the quantity of water to be measured and the conditions controlling such measurement. One engineer, having in mind the measurement of streams in which the maximum discharge does not exceed 10 sec-ft., might advocate the use of the Cippoletti weir; the other, having in mind the measurement of the flow of the Mississippi at New Orleans, might as strongly advocate the use of the current meter.

When adequate data are obtained, reports will be prepared presenting plans for the control of floods in particular drainage basins. Discussion of such reports will be of great value, in that it will concentrate the attention of engineers on specific problems. With this concentration of effort, in solving one problem at a time with but one set of conditions, no doubt most engineers will come to agreement, and possibly all will agree that under certain conditions a reservoir

Mr. La Rue.

may serve more than one purpose; that in some drainage areas detention basins will solve the problem; that in others reservoirs must be constructed; and that on long rivers it may be necessary to construct check dams, detention basins, reservoirs, and levees.

So far as the problem of flood control and flood prevention is concerned, it would seem that the most valuable service that can be performed by the Engineering Profession at present is that of urging (page 1234) the establishment of a "special agency, supported by adequate appropriations, for the purpose of studying stream regulation in its largest sense, and under whose direction all data shall be collated, according to uniform standards and systems, so that appropriate development of the science shall be made".

Mr. Gannett.

FARLEY GANNETT,* ASSOC. M. AM. SOC. C. E.† (by letter).—After reading the report of the Special Committee on Floods and Flood Prevention, the thought presents itself: "What is to be done next"? It was hoped that the Committee would indicate a definite course of procedure to be followed in order to accomplish the gradual reduction of the flood hazard, but it seems that it has not done this. Some members of the Society doubtless expected to find in the report tables showing the magnitude and frequency of floods, together with flood damage estimates, and other information, but it is thought the Committee was wise in not publishing these data. Such data are most useful and necessary to accomplish the control of floods, and their wide distribution would facilitate the intelligent construction of such works, but, without a bountiful appropriation, the Committee could not have been expected to collect, correct, arrange, and tabulate the vast store of records which it could have found available. A list of references to published sources of flood information, however, would have been a useful addition.

Flood data should be collected, studied, analyzed, revised, and made available to the Profession; and would not such work be a proper avenue for the expenditure of the funds of the Society? Certain States are undertaking to do this; the United States Government, in several different Bureaus, is doing some work along these lines; individual engineers have done a great deal, but they have not the resources in time or money to treat the data in the proper way; and, therefore, it seems that, if this information is to be placed at the disposal of the Profession, the American Society of Civil Engineers is the proper organization to do it, even though it may cost many thousands of dollars. It is recognized that there are other fields which probably deserve equal attention by the Society, and that all desirable fields cannot be covered simultaneously, but is there a reason for doing nothing simply because all cannot be done at once?

* Harrisburg, Pa.

† Now a Member.

Mr. Gannett

The Flood Committee might have been expected to publish flood height and damage data or to recommend concurrent State and National legislation which would tend toward reducing the flood hazard. The latter is believed to be by far the more important. It is one thing, however, to recommend remedial legislation and another thing to have it adopted, and that is where the real work would come.

Channel Reduction by Encroachments.—The writer must disagree with the Committee in its statement that "the obstruction of the flood plain by the works of man has in many cases largely increased flood heights for the same volume of discharge". In this discussion we are dealing with great and damaging floods on large streams, and, as a result of an intensive study of past floods and experience in passing upon proposed encroachments in stream channels, in enforcing the laws of the State of Pennsylvania in this respect, the writer thinks that, except for small floods which are confined within well-defined channels, river bank encroachments, and obstructions, have, with but a few exceptions in Pennsylvania, had small effect on flood heights. When floods are confined to channels between normal banks, their height is raised by bank extensions, bridge piers, buildings, foundations, etc., but great floods, in Pennsylvania at least, almost everywhere leave the well-defined channel and inundate the whole valley from hill to hill. During such floods, which overtop the river banks, the extension of such banks makes little or no difference in the gauge height attained.

With small streams the statement of the Committee applies more truly, however, because railroad embankments, highways, etc., crossing such streams over culverts of insufficient capacity, with long, high approach fills, often retard the flow and back up the water to damaging heights. When such accumulations of water finally overtop the embankments and cut them away, the damage and stage below are often far increased.

On Mill Creek, at Erie, Pa., with a tributary water-shed of 13 sq. miles, $2 000 000 damage was done by a flood on August 3d, 1915. This was the third great flood during the last 40 years, each doing vastly more damage than the one previous, not so much because there was more water, but because each time there was vastly more property, in houses, stores, and factories, to be damaged, and also because inadequate culverts carrying streets over the creek, with approach fills, became clogged, backed up the water, were overtopped, and the approaches cut away, to disgorge the accumulated water down to the next culvert, where the process was repeated.

One of the greatest obstructions which can be placed in a big river, as far as impeding the area is concerned, is a masonry arch bridge. Yet when the Cumberland Valley Railroad Company asked

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permission to place such a structure nearly a mile long over the Susquehanna River at Harrisburg, it was found that its effect on an estimated maximum flood of 900 000 sec-ft. (previous actual maximum flood 700 000 sec-ft.) would be to raise it 0.9 ft. over what it would be without a bridge.

Encroachments, however, are going on in every place where no control over them is being exercised; and, if permitted to continue, will in time become of consequence. Heretofore, except at a certain limited number of places (considering only large streams), they have not become of damaging dimensions, but, as every little bit tends in the wrong direction, all encroachments which would tend to restrict flood discharge below safe limits should be prohibited. Few States have legislation on this subject. Until recently, the National law regarding the building of structures in navigable rivers was not enforced from this point of view. The length of rivers classed as navigable is so great, and the corps of engineers available to the War Department to enforce this law is so small, that an attempt to carry out its provisions is only made at certain points.

Pennsylvania has had a good law, well enforced, for 9 years, controlling encroachments along, or obstructions in, stream channels, and a vast benefit has resulted, not so much in bettering previously existing conditions as in preventing worse conditions from arising; and improvements over previous conditions have been obtained at many points. This law prevents the dumping of wastes of all kinds over river banks, unless the slopes are protected, and thus tends to reduce silt carriage by the streams, to be deposited lower down. None of the neighboring States has such laws, and there is nothing to protect Pennsylvania from receiving in its rivers the washings of refuse from banks in States up stream.

Such a law, enacted by all States, modified as necessary to meet local conditions, is a prime requisite and a most proper recommendation for the Flood Committee to make, in order to preserve the streams as they are, prevent them from getting worse, as far as channel capacity is concerned, and to effect a measure of improvement whenever opportunity offers.

The Popular Fear of Dams.—Another thing which the Society can do to further flood control is to assist in disabusing the public mind of the idea that a dam is a dangerous structure. As a result of several disastrous dam failures, two of them in Pennsylvania, the public has the idea that a dam is bound to be a grave source of danger, and the location of such a structure above a city a menace. The broadest schemes of flood control cannot be developed unless this feeling is removed, and it can only be removed by a concerted campaign of publicity such as could and should be carried on by the American Society of Civil Engineers.

Mr. Gannett.

Surveys and estimates at Erie showed that the most satisfactory as well as the most economical method of controlling floods in Mill Creek was by the construction of a detention reservoir and a conduit 18 ft. wide; but, on account of the feeling in Erie against a dam of any kind, and of the necessity of going before the people with a bond issue election for funds with which to construct the flood control works, it was decided to adopt an alternative plan of a much larger conduit and omit the dam entirely, at an added cost of more than $100 000. It was believed by the City Commission that if a plan involving a dam was submitted to the voters the bond issue would fail. Without the dam, the bond issue of $950 000 carried by a five to one vote. The City Commission of five members was unanimously in favor of the plan involving a dam, as opportunity was afforded to convince them of its advantages by frequent discussions; but such methods could not be applied to the entire population in the short time available. Therefore the furtherance of a propaganda favorable to dams, and the spreading of the conviction that dams are not dangerous, would be an important function which the Society could assume in furthering flood control.

Multiple Use of Reservoirs.—The writer believes, with Gen. Chittenden, that reservoirs may be utilized satisfactorily for flood control and other purposes as well. He had occasion to work out this problem on two large storage projects, and in one of these cases it was thoroughly proved that a reservoir, designed primarily for increasing the dry-weather flow for industrial use, would reduce floods in the congested section of the river below, by 30% in height, and would practically eliminate damage. In the other, by raising a proposed masonry dam for water power a very few feet, almost complete control of floods on a water-shed of 1 200 sq. miles, would be effected. In the former case, the reservoir is long, wide, and shallow, covering 30 sq. miles, on a water-shed of 150 sq. miles. The absorption of flood flow by this reservoir would be enormous, a 3-ft. rise absorbing an 8-in. rainfall on the water-shed. The freeboard on the dam is 10 ft. In the latter case, the dam is about 260 ft. high, and the reservoir more than 40 miles long. The capacity curve of this reservoir is such that a few feet on top gives capacity equal to a large percentage of the volume below the level required for water power.

With few exceptions, storage in the upper part of a reservoir is the cheapest, so that, by increasing the height of dams erected for other purposes to a height greater than necessary for that primary purpose, the necessary storage for flood absorption will be obtained at more reasonable cost than if a dam or reservoir is built only for flood control.

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Several of the European nations have in this way created effective control and utilization of important streams. Thus, storage developments on the Oder River, in Silesia, has effected flood protection in the valley below and has also permitted the generation of several thousand horse-power; storage for navigation improvements on the River Rhône has reduced flood heights below storage reservoirs constructed with both ends in view.

Floods come so seldom, strike at such widely different parts of the land, and at such varied seasons, that in the popular mind they have come to be accepted as acts of Providence. This superstition is being gradually eliminated, but the feeling is still prevalent that floods, like lightning, never strike twice in the same place, and so there is no use trying to do anything about it. Furthermore, the writer thinks most engineers will agree that it is impossible to construct flood control measures, where a likelihood exists, if a damaging flood has not occurred, that the financial and governing bodies will refuse to "lock the barn door before the horse is stolen," even though one can show the presence of thieves in the vicinity and the absence of locks on the doors. Flood protection or control measures are usually very costly, and floods seldom recur in any one place, and it is impossible to prophesy how soon the next one will come, so that it is impossible to determine exactly the amount of money which can be expended economically for such works. Again, it is usually necessary in large flood control projects to obtain the co-operation of corporations, cities, counties, States, and the Federal Government, or several of them. All these conditions, as well as numerous others, make the final accomplishment of water storage measures, for flood control alone, a long and laborious undertaking, so long that often the necessity therefor and the losses and horrors of the last flood have been measurably forgotten before all these steps can be accomplished, and the project fails.

For these reasons the writer believes that real widespread flood control through reservoir construction and storage will be brought about by the construction of reservoirs principally for other utilization purposes, such as water supply, water power, industrial use, navigation, irrigation, and for esthetic purposes, such as park lakes, improvement, sanitation, etc.

Our existing State laws, under which most works of this kind would be and are built, will have to be amended in most States to provide a means by which such multiple use may be made of reservoirs through the division of cost between, for example, a corporation building a water-power reservoir and a city desiring flood control, or between one city desiring water supply and another requiring flood control, etc.

Mr. Gannett.

Such legislation has been suggested for Pennsylvania, but has not yet been placed on the statute books. Projects involving several hundred thousand continuous horse-power are now chartered by this State, but remain unbuilt, because of the lack of definite and favorable water-power laws under which they may be built and operated. Others are too costly, as water-power projects alone, to warrant construction in this State, the home of coal. With favorable water-power laws, and with the right to co-operate with municipalities, counties, and the State, many hundreds of thousands of horse-power, now going to waste, would be developed; many towns and cities would have reduced flood hazards, navigation on several large rivers would be improved, and sanitary conditions along the water-fronts of many river communities would be greatly benefited.

Thus another line of legislative enactment, essential to the promulgation of flood control, is the passage of laws permitting and facilitating the generous development of water supply, water power, and irrigation projects, and the division of the cost of constructing and operating them between corporations desiring such works for financial return, and municipalities, counties, States, and the Federal Government desiring them for flood control and navigation.

The Water Supply Commission of Pennsylvania approved of the incorporation of a large water-power project in the western part of the State, involving the construction of large storage reservoirs and the installation of 200 000 h. p., on condition that the operation of the main storage reservoir be subject to the direction of the Commission, as respects maintaining a certain dry-weather flow below it, and as respects absorbing floods.

Forests.—The writer thoroughly agrees with Gen. Chittenden in his contention that, though forests may and probably do mitigate ordinary floods to a certain extent, they do not affect the volume of discharge in great floods. Anything but circumstantial and theoretical evidence on this point is impossible to obtain. The studies which were made of Pennsylvania river floods showed that whenever we could go back 75 or 100 years, substantial and reliable records of floods were often found exceeding anything in more recent years, since the forests were largely removed. This was clearly shown by finding accurate records of two floods at Pittsburgh, in 1762 and 1763, considerably exceeding that of 1907. Also by finding records of a flood on the Susquehanna River at Wilkes-Barre, in 1865, exceeding that of 1902 by several feet in height, and floods in 1784 and 1807 nearly as great. A flood in the Schuylkill River in 1850 exceeded that of 1902 materially. These facts prove nothing, but they show that the dense forest growth, which covered the State at the earlier dates, did not prevent great floods, which, if they occurred to-day, would be truly disastrous.

Mr. Gannett.

Detention Reservoirs.—The writer cannot but feel that on large streams, where water power might be obtained, or where navigation improvement might be gained by storage, detention reservoirs are an economic waste. It is unfortunate that the greatest and most spectacular flood control project of our time, the Dayton project, is planned on that principle. The conditions there apparently made that the only plan that could be adopted, just as at Erie, conditions made the plan without a dam the only one that could be put through. The Erie case also is unfortunate, but being smaller, the failure to utilize the best and most economical method is not so harmful, as it is not so widely known and will not be pointed to so frequently as a precedent as will the Dayton project. The failure to utilize the power of the Scioto River and its tributaries, which, with permanent reservoirs of greater capacity, might have been made available, will set a precedent which will undoubtedly lead to the utilization of this plan elsewhere, where the conditions are not the same and where complete utilization and conservation would have been practicable.

Flood Warning.—The worst feature of floods is the damage they do to life and property. There are two ways of reducing this damage, one is to control the floods and lessen their height, and the other is to get out of the way of them and remove perishable property to places above flood height. The first proposition has been much discussed, but the latter, it seems, has not received the attention it deserves. This involves flood warnings. By timely warnings many lives can be saved and much valuable, perishable property can be removed beyond the danger zone. Until the far-distant day when flood control is general, more attention should be devoted to early and accurate flood warnings.

The Federal Government maintains a flood warning service which has proved itself of great value. It is a Herculean task, however, to send flood warnings over this vast land of ours, and the means at the disposal of this bureau have confined its work to certain points and a limited number of streams.

Following the flood of March, 1913, which visited streams in Western Pennsylvania, a law was enacted, and $10 000 was appropriated by the State Legislature, with which to give to the people a more intensified flood warning service than was possible by the United States Government. This law was drafted with the approval of a United States Weather Bureau representative, and in the 3 years during which it has been enforced by the Water Supply Commission, a great quantity of property has been saved from damage. A larger appropriation would make greater savings possible. In Pennsylvania, however, this service is hampered by the lack of similar service on the head-waters of the Susquehanna, Allegheny, and Delaware Rivers,

which rise in New York and pass through or along the border of Pennsylvania for hundreds of miles. Mr. Gannett.

Either by vastly increasing the resources of the United States Weather Bureau for flood warnings, so that it could give attention to other than the few great rivers of the country, or by adopting uniform flood warning legislation in the several States, much loss in life and property could be prevented while we are waiting for control works to be built.

An important duty of such a service is also that of warning prospective constructors of railroads, highways, bridges, factories, mills, and dwellings of the limits of the flood zones of rivers, so that they may not locate within such area, or if they do, that the building site may be elevated above flood level. If there were such an organization in each State, under State or Federal control, and the people knew of its existence, as they would if proper publicity was given to it and its work, then it would be possible to reduce greatly the increase in the damageable property which is constantly being placed in the way of floods.

In conclusion the writer would suggest that it is the province of this Society to do three things:

1.—Place at the disposal of its members full and accurate flood data;

2.—Change the popular fear of dams into an understanding of their security;

3.—Formulate and push through the adoption of legislation along these lines:

A.—To control obstructions and encroachments in or along streams;

B.—To permit of co-operation between corporations, States, municipalities, and the Federal Government in the construction and operation of storage reservoirs for water power, water supply, and other commercial uses, so as to adapt them for flood control as well;

C.—To extend the Federal, or establish a State, flood warning service.

C. E. Grunsky,* M. Am. Soc. C. E. (by letter).—The discussion which the Committee's report has provoked is of a character fully justifying the report, notwithstanding the fact that the Committee was not in a position to attempt to say the last word on the subject of flood control. Mr. Grunsky.

Some of the statements made by those who point out the incompleteness of the report merit reply, in order that wrong impres-

* San Francisco, Cal.

Mr. Grunsky.

sions may be corrected. Mr. Eakin says: "It is indirectly stated that the use of levees effects an increased channel storage that induces a lowering of flood levels." No one who knows anything about the propagation of a flood wave could by any possibility intentionally make the statement that the reduction of the flooded area which results from the construction of levees will reduce the elevation to which water will thereafter rise between the levees. The language used by the Committee (of which the writer was a member) is therefore unfortunate. The levees are put on the banks of rivers for the purpose of confining the water to a restricted channel and thereby increasing the rate of flow in this channel, and this cannot be done without causing the water between the levees and within the channel to rise to greater height than that which would have been attained without levees. It takes the crest of a flood wave 10 days, or about that time, to travel in the Mississippi River from Cairo to New Orleans. Let it be assumed that at flood the volume of flow is such that levees confine the water to the river and to bank-land areas between the levees. It is evident that in such a case the discharge of the river in excess of ordinary flow for about 10 days before the cresting of the flood at New Orleans has been consumed in filling the river channel above that point from a low to a high stage. It is also evident that this channel storage is greater than it would have been if there had been no levees. To the extent that this increase of channel storage is made possible by the levees it is a factor making for a reduction of the maximum flow at all down-stream points.*

Channel storage between levees does not reduce the total run-off, and it does not reduce the water surface elevation of the river at flood, unless this be in the exceptional case of scour, which may result from the higher velocities in the channel between levees than existed under natural conditions. No claim can be made, and none was intended to be made by the Committee, that the confinement of water by levees will decrease the original maximum stream flow at the mouth of the river. Levees are built for the purpose of reducing flooded areas. They are built to keep water away from land that without them would be less desirable for agricultural, industrial, and other uses. Consequently, for the same volume of flood flow, the opportunity for storage, taking the river basin in its entirety, is less after levees are built than before. The flood wave reaches the mouth of the leveed river with less elongation than under natural conditions, and consequently the discharge at the peak must be greater. As an illustration, the Sacramento Flood Control project may be referred to. Under natural conditions the floods of Sacramento River inundated broad lateral basins, and the lower reaches of the river had a maximum

* See *Transactions*, Am. Soc. C. E., Vol. LXI, p. 332; and also Report of Commissioner of Public Works of California, 1895, p. 130.

flow of about 200 000 sec-ft. The withdrawal from inundation of the islands of the delta region of this river and of the various flood-basins which border the river will increase the momentary flood maximum to more than 500 000 sec-ft. Of this quantity, in the latitude of Sacramento, it is proposed to hold as much as possible in the river, 20 to 30%, confined between levees of reasonable height, but at elevations which at extreme flood will be about 7 ft. higher than the original river flood-plain. It is the additional channel storage represented by this added depth of water which the Committee had in mind when it referred to "the resultant reduction of flood height." What the Committee desired to do was to call attention to the added storage as a factor making for elongation of the flood wave and consequent reduction of maximum discharge, without any intention of claiming that channel flood heights as originally existing would be reduced by the construction of levees. Mr. Grunsky.

In connection with this matter, and in reply to the remarks of Gen. Chittenden, the general principle should be kept in mind that the elongation of a flood wave as it passes down stream is due to the effect of storage. If there were no change possible in the volume of water in storage between an up-stream and a down-stream point, as in the case of a closed conduit, there could be no elongation of the flood wave. The accession of water at the upper point would at once be manifest by an increased discharge in equal quantity at the lower point. Storage, whether in the channel or in overflow basins, has the effect of elongating in some measure every flood wave that travels down a river, and by such elongation the time of passage is increased and the flow at the peak is correspondingly reduced. It follows, as demonstrated by the writer in a paper prepared in 1880 while he was Assistant State Engineer of California, and published in 1895 in the Report of the Commissioner of Public Works of California,* that the maximum discharge of a river which receives no accessions from tributaries will decrease with distance down stream. Every decrease of available storage space, such as may result from a contraction of the waterway, will reduce the effect of storage and will tend to increase the maximum discharge at down-stream points. Every increase of storage space, resulting from the raising of levees or the adding of retention reservoirs, on the other hand, will decrease the maximum flow at down-stream points.

The Committee shares the view expressed by Mr. Eakin when he says:

"No individual or organization has as yet commanded such proficiency in all these sciences as to enable them to outline a programme of river treatment with dependable authority."

* See also *Transactions*, Am. Soc. C. E., Vol. LXI, p. 332.

Mr. Grunsky.

Herein lies the justification of the Committee in presenting a report in which the shortcomings were known to the members, and which was not put forward as exhaustive on the subject of flood control, a subject which necessarily presents as many different aspects as there are river systems.

It happens frequently, as in the case of the Sacramento and San Joaquin Rivers, in California, that some of the natural flood basins along these streams, such as Butte Basin to the northward of the Marysville Buttes, Upper Colusa Basin on the west side of Sacramento River, and, to a lesser extent, Sutter Basin, Lower Colusa Basin, and American Basin, and some of the submersible lands along the San Joaquin River, partake of the nature of the lands which in connection with flood-control projects on other river systems are selected for detention reservoirs. The demand is strong in the California case for the protection of these submersible valley lands against overflow. It becomes an economic question, then, to determine where the line should be drawn, and it is by no means certain that complete protection against floods is in all cases justified. Occasionally, it will be found advisable to retain a natural overflow basin, not only for the control of the floods which it helps to effect, but also as a recipient of some of the silt carried by the stream. The permanent utilization of a natural flood basin seems particularly desirable when the basin is situated so that, while acting as a detention reservoir, it can be drained of its water in time to leave the land available for the cultivation of summer crops. In 1850 the United States gave to Arkansas and a number of other States, for the purpose of reclamation, the land which was segregated as swamp and overflow, and these several States thereupon proceeded to divest themselves of ownership as rapidly as possible. They blundered. The land subject to inundation should have remained in public ownership until comprehensive plans for flood control were made. It would then have been a simple matter to have the plans carried out in proper sequence, and much embarrassment would have been avoided that now results from the demand that protection be provided for lands which, if they were not in private ownership, would to no inconsiderable extent be put in a class requiring only partial protection or fair protection against ordinary and not extraordinary floods.

Mr. Hill points out that the extreme floods of recent years are not due to changed climatic or physical conditions. The example which he supplies of an extreme flood stage in the Great Miami in 1805 before forests had been removed suggests that it would not be out of place to refer to some additional facts to show that a connection between deforestation and stream flow will remain difficult to establish.

Mr. Grunsky.

The writer, in a paper presented to the Society some years ago,* has shown that the dominating cause of changes in the surface elevation of the Great Lakes is the climate, and that a long period of less than normal rain and snow may be followed by another with more than the normal. A still better example to illustrate the variation in the rainfall and run-off is the Great Salt Lake. This lake was very low when visited and sounded by Capt. Stansbury in 1849. It was low when visited by Fremont some years earlier. In the years preceding 1869 it rose, ultimately reaching an elevation about 11 ft. above that of 1849, and then, with numerous minor fluctuations, decreased in extent and in elevation until 1906, when its water surface was about 4 ft. lower than it had been in 1849. It was supposed by many that, on account of the increasing utilization of water for irrigation on its water-shed, the lake would never again rise to anywhere near the high stages of 1868 to 1876, but this view was in error. The lake is now about 8 ft. higher than it was at its lowest stage in 1904, having maintained this high stage for several years. For a long time preceding the low period of 1849, and extending to about 1861, the rainfall in the region tributary to Salt Lake as a water-shed must have been less than normal. Then followed a number of years in which, taken collectively, there was more than normal rain, until about 1876, after which there came another period with less than normal rain as the average for a number of seasons. The consequent decrease of run-off and the increasing use of water for irrigation brought the lake to its lowest stage in 1906. In the case of this lake, which has no outlet, the rise and fall is a fairly good index of the run-off, and therefore of the rainfall, when groups of consecutive years are considered. The problem, moreover, is not here complicated by deforestation. It appears from this and numerous similar cases that very large variation in the seasonal water production of any region may be anticipated, and that the persistence of any tendency which the seasonal run-off may have to increase or decrease for a number of years is not to be accepted as conclusively demonstrating either a permanent change of climate, nor yet that such increase or decrease is in any way related to aforestation or to deforestation of water-shed areas. The same general statement will apply also to the possible momentary maximum stream-flow. The factors affecting the rate of run-off are, moreover, so many and so inter-related that it will remain difficult if not impossible to make a conclusive determination of the effect of forest growth on flood discharge.

Mr. Leighton is evidently of the opinion that no report on the subject of flood control can be a well-considered report which does not

* *Transactions*, Am. Soc. C. E., Vol. LXIII, p. 31.

Mr. Grunsky.

recognize the forest as one of the material factors which influence the magnitude and frequency of floods. Much as the writer has studied this subject, he has yet to find any convincing demonstration that there is such an effect. Generally, the individual cases cited are so complicated by related circumstances that the evidence is not conclusive. The writer is not prepared to accept, and does not believe that the Profession will accept, as final, any mathematical demonstration that forest growth will markedly reduce or meliorate flood conditions. If the statement made by the Committee gives the impression that the quantitative influence of the forest on the frequency and magnitude of floods is not proved, this is exactly what one member of the Committee, at least, intended. This is without prejudice to the forest, which the writer would like to see extended and maintained to meet fully at all times the requirements of the country. The writer, moreover, does not wish to be classed among those who will not concede that it may yet be possible to demonstrate the true relation of the forest to the flood; and, for that reason, he subscribed to the statement in the report that the exclusion of the forest as a method of flood prevention had not yet been demonstrated, though he believes that, except in the rarest cases, conditions will never be such that the forest will become a material factor.

The maximum stream flow is not dependent solely on the rainfall and the rate at which snow is melting; it depends also on the condition of the surface of the ground on which the snow lies or the rain falls. Usually, the period of intense rain which produces flood stages is preceded by rain which saturates more or less thoroughly the top layers of the soil. This is least likely to occur in sandy and gravelly regions and where loamy soils are well tilled and are deep. It is most likely to occur in regions where the ground is swampy and naturally full of water, or where the flood-producing rain falls on frozen ground which requires but little water for complete saturation. It follows that, whenever a region which in its natural condition was swamp has been made arable and brought under cultivation, the probability of complete saturation preceding a flood-producing rain will have been reduced, and some effect on the frequency and intensity of the flood condition in such a region will have to be conceded to the reclamation work. The main effect of any such modification of surface conditions should be sought in terms of water storage and retardation of the flood wave, because, if thereby the resultant elongation of the flood wave can be determined, some basis will exist for estimating the quantitative effect on the maximum flow. The writer agrees with the comments of Gen. Chittenden in this regard, and shares his view that the modification of the effect of soil cover and absorption by Man's occupancy and use of the soil will be a negligible factor in the excessive rainfalls which produce great floods.

Mr. Grunsky.

In the light of present information, the writer agrees too with Gen. Chittenden when, after referring to the aggravation of flood damage which results from the obstruction and use by Man of Nature's overflow channels, he says:

"The foregoing remarks disclose the reason why flood destructiveness is increasing. It is not because floods themselves are increasing, either in frequency or intensity, but because property subject to destruction is very greatly increasing, and as yet protective work does not keep pace with this increase."

The writer, though having qualifiedly subscribed to the Committee report, is not in full accord with its reference to outlets. Concentration of the flood flow in one channel is desirable. Separation into a number of channels is undesirable. The conditions, however, are not always such that a single channel can be given adequate capacity to carry all the water presented at flood. The study of the question what to do when this is the case led the writer to formulate certain principles for the treatment of Sacramento River floods,* substantially as follows:

First.—Make the river carry as much water as possible between levees of reasonable height.

Second.—Let the flood-water in excess of capacity between levees go out of the channel at selected points under control.

Third.—Confine the outgoing surplus waters to limited areas, and keep them under control until they may re-enter the river or can be delivered into the bay.

The writer has not modified his views in this respect, having, subject to these principles, always been in favor of relief weirs or controlled outlets. These are not to be confounded with ordinary outlets which begin to function before the river has reached the danger line and which become secondary branches of the stream. It is outlets of the latter kind, in the sense of a division of the stream, which with good reason are disapproved by the Committee.

The writer is pleased to find Gen. Chittenden in accord with the writer's view that the cut-off is generally a desirable improvement, and that the fear that shortening a river channel will interfere with its usefulness as a commercial highway is not ordinarily well grounded. The sketch, Fig. 5, is from an old document, and shows the extent to which, by intelligent direction and restraint, the course of the Rhine in the vicinity of Germersheim was modified during the period from 1817 to 1861. This sketch shows channel shortening and alignment modification quite comparable to that which might be carried out on the Mississippi, on the Missouri, or on the Colorado, if the necessary outlay, including permanent bank revetment, were justified, and the occupancy and use of the bank-lands do not already set a

* Report of Commissioner of Public Works of California, 1894–95, p. 59.

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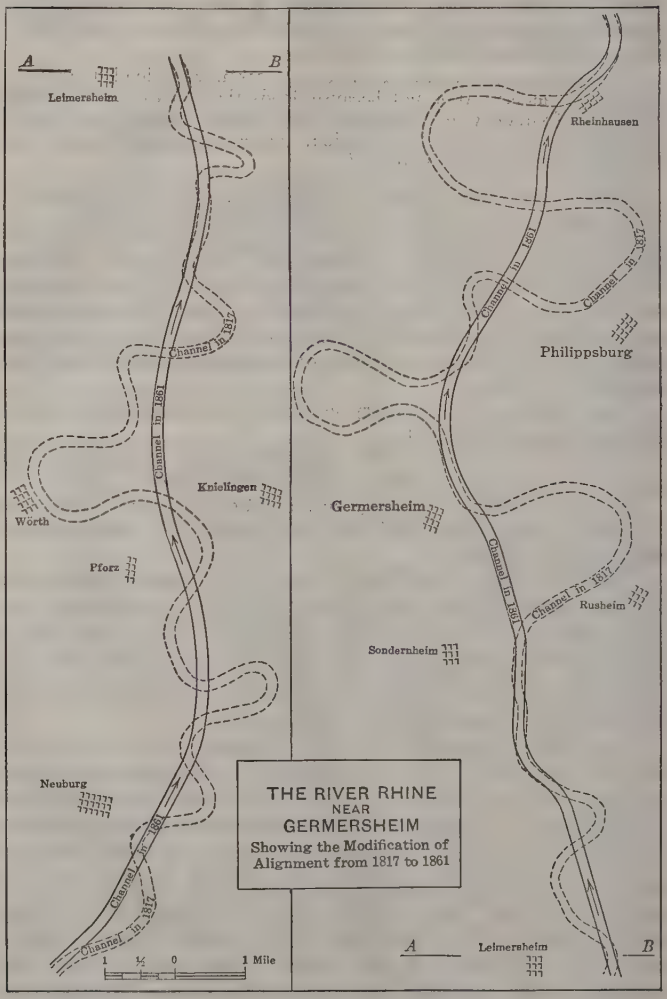

FIG. 5.

limit to the alignment corrections which might otherwise ultimately be hoped for. Mr. Grunsky.

In the case of the Rhine, the 1817 channel length of 63.5 miles, covered by the diagram, was reduced by cut-offs and corrections of alignment to 27.6 miles. The river was shortened in this stretch by 46% of its original length. Though a stream of considerable magnitude, having a width of about 800 ft., the channel control was effected successfully by excavating guide channels, by constructing training walls of fascines and brushwork, by obstructing the old channel with dams and dikes and, finally, by bank revetment and protection wherever the river in its meanderings reached the prescribed alignment. The channel improvement of the Rhine was not confined to the stretch shown by Fig. 5, but extended for miles in both directions therefrom—up stream as far as the Swiss boundary and down stream to below Mannheim at the mouth of the Neckar.

C. McD. Townsend,* M. Am. Soc. C. E. (by letter).—In submitting a closure to the discussion of the Report of the Special Committee on Floods and Flood Prevention, the writer has been impressed with the force of Bishop Warburton's remark that "Orthodoxy is my doxy". There is a tendency by various members of the Society to criticize the Committee for not giving special emphasis to the particular "ism" they advocate, without giving due consideration to the difficulties of formulating any report on which the Committee could agree. Mr. Townsend.

The subjects which were before the Committee have been discussed for many years, and most of its members had positive convictions which, if each had insisted on expressing, would have led to as many reports as there were members. A series of such essays would not have had much weight with legislators or the general public. Mr. Leighton lays particular stress on the necessity of engineers—and particularly of this great National Society of Civil Engineers—influencing legislation. The only way such results can be obtained is by united action. It became necessary, therefore, for the Committee to seek some common ground on which its members could agree. It was the realization of the necessity of adjusting divergent views, if results of value were to be obtained, not the shirking of duty, which led the Committee to make the safe and conservative report which Mr. Leighton criticizes.

Others criticize the report because it does not discuss the subject in sufficient detail. This phase of the subject was considered by the Committee, and it was found that the territory is so vast, the problems arising in various sections of the country so different, that either general principles, alone, could be enunciated, or a treatise of interminable length, based on incomplete data, would have resulted. A

* St. Louis, Mo.

Mr. Townsend.

brief report, therefore, was agreed upon, leaving to the members the privilege of elaborating, in the discussion, such details as they considered advisable.

The science of river hydraulics is not in such a nebulous condition as Mr. Eakin appears to think, nor does the river engineer have to be guided solely by the classic work of Humphreys and Abbot. Although, possibly, American engineers have not investigated sufficiently the laws governing the flow of rivers, those of Europe have given the subject profound study. The researches of Fargue on the Loire, Girardon on the Rhône, Liliavski on the Dnieper, Hagen, Schlichting, and Jasmund on the rivers of Germany, and others, have revolutionized the science in recent years. Moreover, the Committee was not appointed to write a treatise on river hydraulics, and could assume that the rudimentary laws governing the flow of water in rivers were known to members of the American Society of Civil Engineers. It is evident from the discussion, however, that many do not realize sufficiently the differences between the laws governing the flow of water in pipes, sewers, and other channels having an immovable bed, and those which govern the flow in a stream having a bed which fills or scours with changes in velocity. Van Ornum's "Treatise on the Regulation of Rivers" and the second edition of Thomas and Watt's "Improvement of Rivers" throw considerable light on the subject, and are more suitable sources from which to seek information than a report on Flood Prevention.

In the minority report, a similar assumption is made. Although the majority of the Committee states that the physical data are lacking on many streams to enable them to solve the problem of flood control properly, they are not prepared to admit that "extensive studies are needed regarding the physical and physiographic laws affecting stream loads, rate of transportation and deposition and the variation of these factors with different conditions" before they can solve the problem if the physical data are supplied.

It is so self-evident that, at such times as the ground is frozen and trees are without foliage, these conditions prevent the utilization of reforestation or ground storage, that there is no necessity of making such a statement in a report. The point the majority of the Committee makes is that, if floods occur under such conditions, there is no need of discussing the effects of reforestation or ground storage. It would be just as pertinent to investigate the porosity of the soil under an asphalt pavement when designing a sewer to carry the water which flows over its surface.

It is the writer's opinion that all investigations of the absorptive powers of different varieties of vegetation and the percolation through different kinds of soils are more valuable for studying problems of irrigation and drainage than those of flood control. Though it is con-

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sidered perfectly feasible to measure these elements within a limited area and for a single river, the writer agrees with Gen. Chittenden that a determination for one situation, or season, or stream, would never apply reliably to any other. Moreover, to determine absorption of soils, it is necessary first to measure the run-off. When the discharge of a stream is determined, it is of no particular value to the problem of flood control whether the remainder of the rainfall has disappeared by evaporation or soil absorption.

The writer has recently investigated floods in Kansas, and they afford a pertinent illustration of this principle. During the summer the soil of Kansas provides a reservoir of enormous capacity, having a power of absorption which has been approximately measured by the Geological Survey. When the soil is prepared to receive it, a heavy rainfall will be accompanied by an exceptionally small run-off, but such conditions are of no value in determining flood prevention, as violent floods do not then occur. The floods in Kansas arise either from storms in February or March, when the ground is frozen and cannot absorb moisture, or in summer from a second storm which sweeps over the country within a few days after the reservoir capacity has been exhausted by a preceding rain. The run-off during ordinary conditions does not enter into the problem.

The majority report emphasizes the fact that reservoirs cannot be utilized simultaneously to reduce floods, to regulate the low-water discharge, and to increase the water-power that can be developed. The minority report seeks to minimize the effect of this statement.

There have been constructed at the head-waters of the Mississippi River a series of reservoirs primarily to increase its low-water flow. The practical manipulation of these reservoirs forcibly illustrates the necessity of the remarks in the majority report. Those who have meadows below the reservoirs demand that the flow be regulated so as not to interfere with their hay crop, those who raft logs want sufficient current to assist their business, and those who control water-power want a constant flow; in times of floods, those living above the dams are insistent that the gates be opened so as to reduce gauge heights on the upper river, and those below demand that the gates be closed to protect their property from overflow.

The statement that levees increase the reservoir capacity of the river channel appears to require further explanation. The construction of levees has increased flood heights on the Mississippi River between Cairo and New Orleans from 6 to 10 ft., and, as the levees are several miles apart, there is an enormous increase in the channel capacity during floods in this distance of 1 000 miles. As an illustration, the channel capacity above the overflow stage is given and compared with that of the Roosevelt Dam, but, as Gen. Chittenden states, the channel capacity of a river is by no means comparable in effect

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with that of a reservoir. The slope of the river is an important factor in the problem. The greater the slope the less being the influence of the channel capacity on flood heights; but, under favorable conditions, its influence is very large. Notwithstanding the inflow of tributaries, the maximum discharge of the Mississippi River at Vicksburg rarely equals that at Cairo, nor at New Orleans that below Red River Landing.

The straightening of a river has been a subject of discussion by European engineers for many years. Prior to 1890, German engineers generally adopted the idea that, in improving rivers for navigation, the channel should be given a uniform width, bends should be eliminated, or at least be given as gentle a curvature as practicable, and the river forced to a uniform slope, the natural gentle slope in pools in some cases being destroyed by submerged dikes extending out from the concave banks. The improvement of the Rhine, cited by Mr. Grunsky, and that of the Danube, cited by Gen. Chittenden, are examples of this type of improvement. It was also adopted by the Missouri River Commission for that portion of the Missouri River between Jefferson City and the Gasconade.

Difficulties, however, were experienced with this method of improvement. No matter how much care was exercised, the slopes through the improved reach could not be maintained as designed. At the upper portion, the water surface was gradually lowered, and a bar as invariably formed below it; an improvement in navigation was created through the reach, but the channel, both above and below, had less depth than existed formerly.

In a paper presented to the International Navigation Congress of 1894, on the "Improvement of the Rhône", M. Girardon called attention to these difficulties, citing as an example the portion of the Rhône called the Canal de Miribel.

He states:*

"The works were carried out according to this programme and finished in 1857; the resulting improvement was very evident and the navigation, in place of the continual uncertainty in which this passage left it, found in the canal of Miribel a channel easy to follow and of sufficient depth at any time of year. For several years this improvement appeared to be the only result of the works and nothing indicated that they could have any inconvenient consequences. Nevertheless the bottom of the canal was attacked; lower measurements began to be registered at low water than before the works were executed and these got lower and lower, while at the exit they marked higher and higher; then the foundation of the works, which had been made at low water-level, appeared in the upper portion of the canal and was even uncovered at the mean summer level of the water, while below the same water-level the tow-path was submerged and became useless throughout

* Official Translation of the Congress, pp. 27 to 29.

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the greater part of the year. It became necessary to improve this situation which might at any time become very serious. The tow-path was first raised over half the length, that is the part submerged; then to arrest the movement it was decided to lower the dividing dike and to cut down the dam of Thil as low as possible, in order to allow the greatest possible volume of water to find its way through the false branches, not only in floods but also at the mean water-level. It was supposed that by this means the main branch would be relieved and that by decreasing the volume of water which crossed it its scouring action would cease. The result aimed at has been partly realized; the movement of the profile has become much slower, but has not entirely ceased; and although the volume of water which passes into the canal has become notably less, the incline [slope] which will set up any equilibrium between the resistance of the bottom and the effective force of the current under the new conditions of depth has not yet been attained.

* * * * * * *

"Thus in spite of the improvement realized, in spite of the enormously extensive alterations that the river has undergone while passing between the constant banks, neither regularity in the cross-section has been obtained, nor constancy in the mean depth, nor uniformity in the incline [slope]; and the bed that has been opened remains formed as everywhere else of a series of hollows [pools] separated by ridges [bars], while the profile presents a series of reaches with a comparatively feeble incline [slope] separated by falls.

"Outside the canal the general incline [slope] of the combined streams at the entrance to Lyons, at kilometre 5, where the profiles join, has not sensibly varied; it was and still is 0.81 m. per kilometre, but its distribution has changed considerably. Throughout the canal, from kilometre 8 to kilometre 25, it was 0.883 m.; it is not more than 0.696 m. per kilometre on the other hand above the canal; from kilometre 25 to kilometre 34 it has been raised from 0.75 m. to 0.96 m., and below kilometre 8 to kilometre 5 it has been raised from 0.49 m. to 1.06 m. per kilometre."

The profile, Fig. 6, derived from De Mas' "Rivières à Courant Libre", illustrates the condition.

Similar effects have been observed on western rivers of the United States. In the improvement of the Middle Mississippi below St. Louis the river was straightened for more than 20 miles. As a result, the slope was reduced at low water from above 0.6 to 0.25 ft. per mile, and the low-water surface of the river at the Eads Bridge was lowered about 3 ft. An excellent channel exists through the improved reach, but the slope on the Chain of Rocks, above the city, has been increased, and dredging is required annually immediately below this reach in order to maintain the required channel depth. Similarly, the Kaskaskia cut-off, above Chester, which shortened the river about 6 miles, has a low-water slope through it of 0.23 ft. per mile, but dredging is required annually on the crossings above and below.

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The improvement of the Missouri River reduced perceptibly the low-water plane at Jefferson City, and a cut-off at Camden Bend, which occurred in 1915, has created a bar 2 miles below it over which on November 3d, 1916, during an inspection trip made by the writer, a boat drawing 3½ ft. had to be sparred, the gauge at Kansas City reading 7.8 ft. At the bend above Omaha Mission, about 764 miles above the mouth, during the June flood of 1916, another cut-off occurred, and on October 20th the snagboat *McPherson*, drawing 3 ft., had to be sparred over shoals, both above and below this cut-off, the Omaha gauge reading 6.4 ft.

On the Arkansas River, a very forcible illustration of the effect of cut-offs was also afforded during the flood of 1916. In prehistoric times the Arkansas River, by the caving of its banks, broke into the

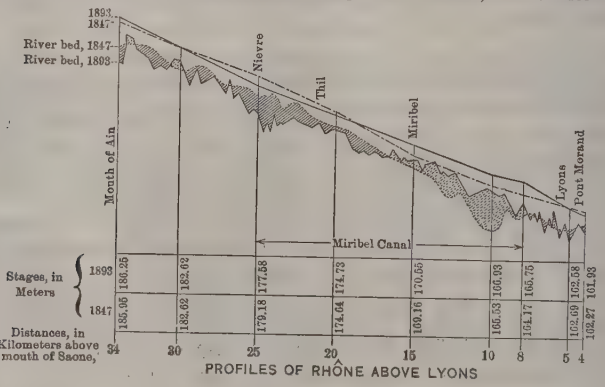

FIG. 6.

White River, about 10 miles above its mouth, and, until this season, the low-water discharge of the Arkansas emptied into the Mississippi through the mouths of the White. The cut-off between the two rivers gradually increased in length, and a sharp bend developed. During the flood of 1916 a cut-off occurred across this neck, resulting in such a shoaling in other portions of the old channel that at the present time (November, 1916) the Arkansas River has resumed its old outlet into the Mississippi and now discharges its low-water flow in a separate channel from the White River.

On the Red River numerous cut-offs have occurred, with results equally disastrous. Near Duke's Plantation, below Garland, the cut-off of 1915 created a bar below it over which at the next low water the depths of the navigable channel did not exceed 1 ft., and the efforts of the river to increase its length by caving after these cut-offs occurred

have been destructive of the levee line which had been constructed along the river banks. Mr. Townsend.

It is the raising of the river bed, not the increase of the discharge, which raises the water surface below cut-offs. The French engineers have called attention to this, and their views are now generally accepted in both Germany and Austria.

In his work on the improvement of the Rhine, Mr. Jasmund discusses the straightening of that river (cited by Mr. Grunsky) and explains that it is due to a previous undue reduction of slope, in former attempts to improve the river, that renders these cases exceptional, and conveys the impression that he disapproves of cut-offs on other portions of the Rhine.

The straightening of the Danube River in the vicinity of Vienna, quoted by Gen. Chittenden, did not improve the navigation of the river as much as was anticipated. About 3 500 000 cu. m. of material were scoured out of the cut, and deposited in the river below, raising its bed, and extensive work has since been required to adjust the low-water channel to these new conditions. It has even been necessary, in order to stabilize the bars, to create in the low-water channel new sinuosities to replace those cut out in the original improvement.

The view that a sedimentary stream seeks to regain its length after a cut-off did not originate on the Mississippi River, but is a corollary to views advanced by Guglielmini more than 200 years ago from observations he then made on the Po. Observations made recently by the Mississippi River Commission confirm this theory. Guglielmini's general principle, that there is a relation between discharge, slope, and the character of the soil, has also been verified by the changes which have taken place recently in the Atchafalaya and Illinois Rivers. In both these rivers a change in their discharge created by artificial means has been accompanied by a marked change in the radius of curvature of bends. A citation of an unreliable reconnaissance by Lewis and Clark is not considered sufficient proof to overcome the overwhelming evidence that can be cited to prove that rivers do strive to adjust their length to their discharge and slope.

Gen. Chittenden also takes exception to the distinction made by the Committee between streams created by glacial action and those carrying a large quantity of sediment. Although not agreeing with Mr. Fuller, that there is a necessity for a new variety of engineer, called by him the geological engineer, the writer believes that all engineers who investigate river problems should have knowledge of geology. The methods of improvement which are successful with the gentle slope of rivers created by glacial action have frequently failed when applied to streams carrying large quantities of sediment. Thus, on the rivers connecting the Great Lakes a permanent channel of 21 ft. can be dredged, but on the Mississippi a channel dredged to 9 ft. fills annually. The permeable dikes, so successful on the Middle

Mr. Townsend. Mississippi, were failures when applied to the Upper Mississippi above Keokuk. On the Illinois River a cut-off could be made with impunity, but the writer believes it is impossible to improve the Missouri River permanently unless cut-offs are prevented.

In reference to outlets or spillways, the writer does not apprehend the danger to the channel of the main river from their construction that some do. He is a firm believer in the idea that rivers have a tendency to correct man's errors, and that, if outlets are constructed, a fill will occur in the outlets rather than in the river channel; but, if the outlet is given such dimensions that it begins to enlarge, there is imminent danger that the main channel will not only fill but be abandoned, unless the process is checked. A river exhibits an objection to serving two masters.

Mr. Knowles. Morris Knowles,* M. Am. Soc. C. E. (by letter).—In contributing to the closure for the Minority, the writer expresses the conviction that the character of the discussion presented demonstrates conclusively that the attempted premature discharge of the Committee, on two occasions, and before a final report was submitted, was an unfortunate mistake. It is evident, also, that the discussion has fully justified the contention of the Minority that assertive statements, with a limited point of view only, and in the face of present divided opinion on the subject, will not promote progress or the development of a much needed comprehensive programme for stream control, but will rather intensify discord where it is not necessary. The object of the Committee, in so far as it could not agree on a final pronouncement, ought to be to promote further investigation and discussion, and not to attempt to close the door by dogmatic expression of a single point of view.

It is apparent, at a time when the importance of regulation of stream flow in its largest sense, which includes flood control as a part, is engaging the attention of the country and impressing itself on laymen, that the Society has lost a wonderful opportunity to promote sound thinking and the development of a programme looking toward wise legislation. It is plain from the public interest, general discussion, and proposed laws, that no one type of control is a universal panacea, or will do for all classes of streams; also, that the opinion of a single branch of scientific men, not in sympathy with all phases of stream regulation, will long satisfy the country, for such opinion will stifle progress and unduly lengthen the period required for reaching the desired results.

The writer expresses the hope, therefore, that, at some later date, the Society may take up this important subject again and continue its consideration until it becomes possible to prepare a report which will merit the consideration justified by the high standing of the Society.

* Pittsburgh, Pa.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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Paper No. 1401

FINAL REPORT OF THE SPECIAL COMMITTEE TO FORMULATE PRINCIPLES AND METHODS FOR THE VALUATION OF RAILROAD PROPERTY AND OTHER PUBLIC UTILITIES*

WITH DISCUSSION BY MESSRS. J. E. WILLOUGHBY, C. E. GRUNSKY, JOSEPH MAYER, AND W. R. MCCANN.

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* Presented to the Annual Meeting, January 17th, 1917.

PREFACE.

In presenting this report to the Members of the American Society of Civil Engineers, the Committee is fully alive to the fact that the art of valuation is still in formative condition, as is evidenced by the conflicting views expressed, or principles enunciated, even by the higher Courts. That debatable subjects arise frequently, and will continue to do so, is indicated by the record of the Committee's work incident to the rendering of this report, for in spite of the prior knowledge of its members on this subject, and professional experience in it, five years have elapsed since their appointment, in which forty-eight joint meetings have been held, many of them consisting of three sessions, and a voluminous correspondence has been carried on, aggregating thirteen substantial volumes.

Nevertheless, the fact that nine men of widely different training and experience, practising in different professional lines and fields, have been able to come finally to common belief upon most of the subjects discussed—the principles which should control the valuation of normal public utility properties—leads the Committee to hope that this report may be helpful to others, and may serve to clarify this very involved subject, to the common advantage of public service corporations and the public served, by aiding in the establishing of procedure and in the reducing of the uncertainties of valuation and rating of public utility properties.

Where differences of opinion have developed, the conflicting views of the Court have been cited, the effect of the application of different theories indicated, and the course of action most likely to lead to a fair settlement of debatable questions has been outlined.

The Committee bespeaks a thorough and open-minded study of the whole report, by the members of the Society interested in the subject. Only with such a background will its full significance be apparent. The subject is so involved and many-sided that brevity and conciseness are not always possible, and that brief statements may not always be capable of isolation from the context without the possibility of causing ambiguity; therefore, though the Committee has prepared an abstract of its conclusions for the convenience of the readers of the report, it hopes that the members of the Society will base their conclusions and discussions on the main report, rather than on the abstract.

The table of contents follows. Thereafter are given, successively, the abstract, the introduction and other chapters of the report, and finally a detailed glossary.

October 28th, 1916.

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ABSTRACT OF REPORT.

The Committee, in its consideration of the subject assigned to it, has been guided by the belief that stress should be laid on fundamental principles and on general methods rather than on details which necessarily vary under different circumstances; that the discussions, unless otherwise stated, should be confined to normal properties; that valuation should have its foundation in fact; that where there are alternative methods equally fair to the parties affected, the one should be selected which is most capable of being carried out effectively under present laws, as defined by the Courts, and which is most desirable for general use in the direction of uniformity; that recognition should be given to the interdependence of the various elements which enter into the problem; that while major attention should be given to the ascertainment of the cost of creating the property in question and its business, due consideration should be given to intangible values, and that the important item of interest, in this report meaning interest compounded annually, should always be considered.

In the light of this preliminary understanding and of the discussions which ensued, the following conclusions have been reached.

Fundamental Principles of Valuation.

The principles and methods must be such that when properly applied the result will be fair to all parties affected and of a nature that will attract to the service of the public capital necessary for building new properties and for improving and extending old ones.

New Properties.—In the valuation of a new property, if the question of a fair return is at stake, the actual investment in the portion devoted to public use, including working capital and development expense, should be taken as the basis for "fair value"; if the same property just ready for operation is to be valued for public acquisition, the basis of "fair value" should be the actual cost, including the money value of services and other considerations involved; if the valuation is to be used for capitalization, the result should be attained in the same way as the return base, except that all parts of the property should be included; and if for taxation, whether of a new or old property, the result must accord with the laws of the State, or, where not governed by such laws, should, to make it equitable, be fixed at a sum consistent with the valuation of other property for taxation.

Old Properties Under Continuous Regulation.—In the valuation of an old property, operating without competition and from its inception under commission control as to rates and methods of accounting, and assumed to have been entitled to earn sums sufficiently large to provide for all expenses of maintenance, operation and taxation, depreciation allowances, and a fair return on the "fair value" of the prop-

erty, the owner should be compensated in some way for losses that he may have sustained during the early years of operation before the property was tuned up and the business developed, either by including in the valuation the sum of the deficiency of earnings in the early years, with interest, or by allowing higher rates of return in subsequent years to offset the early deficiency.

Old Properties Not Under Continuous Regulation.—In the valuation of an old property which has not been subject to continuous regulation, not only are the foregoing principles important, but there are many cases in which equity calls for the inclusion of not only the sum representing the sacrifice by the owner but also a further positive or negative sum representing valuable property or rights which may have been acquired or lost by the owner as a result of time or through the failure of the public or the owner to assert their authority, the Courts holding that the present value of the property should be used rather than its cost.

Physical Property to be Included in Valuation.

This varies in different cases with the use to which the valuation is to be put and the law governing the case.

Used and Unused Property.—In cases of rate regulation, only the property considered to be devoted to public use should be included, embracing that which is in active use and also that which is properly and reasonably held in reserve to insure the safety, economy, sufficiency, and continuity of service. In valuations for capitalization and public acquisition it is obvious that all of the property in question should be included.

Retired or Discarded Property.—Plant units which have been definitely abandoned and are not likely again to be used, due to having been worn out in service or by reason of the reconstruction of the property, should be excluded from the valuation and should appear either in a separate schedule of retired or discarded property, or such full statement of the conditions should be made as will definitely fix the status of the units or parts. Temporary works necessarily built in connection with, or required for, the construction of permanent works, or for furnishing service to the public at an earlier date than it could be furnished by the permanent works, should be included in the valuation.

Excessive Size or Capacity.—No reduction should be made in the valuation on account of excessive size or capacity, except when the excess is so great as to be clearly unreasonable and is the result of not using proper foresight.

Donated Property.—Lands or other property voluntarily donated to a public utility should be included when determining the reproduction cost, on the same basis as land and property otherwise acquired.

Leased Property.—In the case of leased property, either the property itself or the lease should be valued, as circumstances may dictate.

Title to Property Not Conclusive.—Structures located on land to which the owner of a public utility has no title should be *included* in the valuation of the property where the owner has been required by law or necessity to pay their cost, including in this class also property voluntarily donated; and they should be *excluded* where other public service companies, the public, or the users, other than the owner in question, have been required by law or necessity to pay their cost.

Working Capital.—It is customary to include under the term "working capital" the amount of cash, materials, and supplies, provided for use in the plant, but not yet forming a part of it, and other current assets which are essential for the proper maintenance, operation, and administration of a property. There should be included in a valuation an amount of working capital sufficiently large, not only to meet the usual requirements, but to provide for emergencies.

Securities Owned.—Ordinarily, the valuation of property devoted to public use should not include securities owned, or surplus cash not forming a part of working capital, except in instances where such securities and surplus cash are an offset, in whole or in part, for depreciation deducted from the cost of the property.

Original Cost to Date.

As defined by the Committee, this is the first cost of the identical property units now in use, including overhead charges.

Difficulties.—While much of the difficulty of determining original cost as thus defined in some cases may be removed, especially in the case of short-lived property, it is not feasible to obtain a dependable result where the absence of reliable historic data makes necessary a resort to estimates, as in the case of old properties consisting mainly of long-lived items.

Schedule.—Generally, it will be found necessary to prepare a schedule in the same way that one would be made for determining the cost of reproduction, many adjustments in the records often being required, even under the most favorable conditions, in order to obtain correct results.

Costs—Unit Costs.—When a schedule is necessary, the corresponding costs or unit costs are essential to the completion of the inventory, and where these are unobtainable, as is usually true in the case of property units acquired or created long ago, the ascertainment of the original cost is impossible.

Overhead Charges.—These are recited at length in the section relating to cost of reproduction. They are inadequately reflected in the records, as a rule, and therefore adjustments are required which are largely matters of opinion and speculation, and which, in conse-

quence, make more uncertain the final result thus often destroying its usefulness.

Development Expense.—The expense, actually incurred in connection with the tuning up and creation of the business of a property, should be included as a part of the original cost to date.

Cost of Reproduction.

Theory of Reproduction.—Estimates of the cost of reproduction should be based on the assumption that the identical property is to be reproduced, rather than a substitute property; that while apparent present-day conditions, that would affect the cost of reproducing the property, must be considered in any logical estimate, yet history must also be considered, to determine what is to be reproduced, the conditions under which it is to be reproduced, and how the estimates must be made; that for all those items concerning which there can be no doubt the engineer should use the basis plainly applying, and that for those that are doubtful, or have been questioned, he should present the effect of the use of the different bases clearly, that the determining body may have the data for a wise decision; and that normal present conditions shall determine the prices and methods for doing the work.

Preliminary Work.—The first step in estimating the reproduction cost of a property is such a study of the property and its history as will enable the estimator to make a complete list of all items, lay out a proper financial and construction programme, and fix proper unit prices to the several items of the schedule, after which the property should be divided into groups of items or units which will lend themselves to the depreciation study and accounting, preferably following some well established classification of accounts.

The field schedules and inventories, suitably recorded on appropriate forms, should be based not only on full and complete surveys and inspections of the visible physical property, but they should also reflect a careful historical search of existing records of original conditions, and other reliable sources of information, bearing on items of material or work which entered into or were incidental to actual existing units, special care being exercised to exclude that which is not capable of definite proof, and to limit the speculative uncertainties as far as possible.

Unit Prices.—In determining unit prices a rational sequence of construction should be assumed for the reproduction of a property; and rational assumptions should be made as to the manner of doing the different parts of the work, whether by the forces of the company or by contract. Unit prices based, where possible, on the actual cost of doing similar work, in a similar manner, under similar circumstances, should be determined by persons of experience and sound judgment. They should be based on the normal average cost of work

for a considerable period—say, 5 or 10 years—in order to give stability to the valuation, so that it may be used for a subsequent term of years. In the case of items which are steadily increasing or decreasing in value, the prices adopted should be normal for the time of the valuation. Full consideration should be given to the time allowed for construction, to climatic conditions, and to the effect of any other significant conditions or limitations upon the cost of the work.

Land Holdings.—Separate parcels of land, such as one or two lots, or a tract of not unusual size, not connected with other lands of the utility in a strip or body, and where freedom of choice in selection may be exercised, should be valued on the same basis as other lands in the vicinity, used for general purposes.

Where large tracts have been acquired, made up of a number of separate entire parcels, the history of the transaction should be completely investigated and allowance made in the estimate for costs of vacation of streets, or acquisition of other rights, incident to the use of the property as a whole. Such further allowances should be made, as may seem to be justified by experience, to cover excess costs, over and beyond the normal market values which existed at the beginning of the project, and which were due to fluctuations of price or other causes beyond control. Normal market values of similar and near-by property, as of the time of appraisal, should be the basis, with such additions as are warranted by the investigation in each case.

In the consideration of values of lands, for rights of way for a railroad, an electric railroad, an aqueduct, or other like property, or, lands for a water-works or an hydro-electric reservoir, where freedom of choice is restricted and where all the land must be acquired, radically different methods must prevail.

In the valuation of lands of this class, a clear distinction should be made between:

(*a*) Lands where the entire tract or parcel is taken and there is no element of severance damages;

(*b*) Lands where only a portion of the tract is taken and where the element of severance damages is present;

and the extent of lands of both classes should be shown.

Historic conditions, where ascertainable, should be given due weight in fixing the extent and character of the severance. All elements of value to the seller, including recognition of all damages to the portions of his property not taken, which would prevail in the case of condemnation of the lands under reproduction, should be considered.

A determination of the relation existing between actual recent acquisition costs and the normal market value of the lands out of which the strip or parcel was taken at the time of acquisition is undoubtedly possible in many cases. It would seem that where such relation can

be determined, its use in connection with the normal market values of these or similar lands at the time of appraisal should establish a reasonable cost of reproduction.

The determination of the figure to be used should be based in all cases on full consideration of the present normal market value of the area of land acquired and of other recent purchases by the same or other companies of similar lands in the vicinity or in districts of like characteristics; the damage to the remaining land, not required, due to severance and all consequential injuries; the amount and character of the costs of acquisition and overhead charges; enhanced prices due to active demand; and any other real items of cost which would be included in case of purchase. But no allowance should be included for special values coming after the acquisition of the property on account of its new use or on account of a greater earning power under the new use, or for any other hypothetical "value".

Estimates should be based on prices and values as of the time of appraisal, be they higher or lower than those prevailing at the time of original acquisition. The thing sought in reproduction is the fair cost of acquiring the property as of the date of valuation.

The treatment of the valuation of land holdings is not yet thoroughly crystallized; therefore the valuing engineer will do well to confer with counsel upon the interpretation of past Court decisions and the legal principles which are most fairly applicable to the case under review.

Overhead Charges.—There are certain expenses called overhead charges, inseparable from the construction of any property, which are a necessary and proper part of its cost, but which are not capable of physical identification after the completion of construction work. These expenses cannot be covered in the estimate of "Cost of Reproduction" by the application of specific unit prices; from their nature they attach to the whole or large parts of the property rather than to any particular units.

Among the expenditures which must be provided for, and classed as overhead charges are:

(*a*) Cost of promotion;
(*b*) Cost of financing and securing the necessary capital with which to carry out the enterprise;
(*c*) Cost of organization, including the incorporation and organization of the company, securing of franchises, and other like expenditures;
(*d*) Engineering, including the making of the preliminary investigations and plans, plans for the construction of the entire property, the engineering supervision of all construction and other work involved in the development of a property, except

such direct supervision as may properly be included in the unit prices of various property units, or as specific charge against some particular schedule or group of units;

(*e*) Administration, including salaries for general officers, agents, accountants, clerks, and other assistants, not included in the engineering and legal departments, and all administration expenses;

(*f*) Legal, including salaries and expenses of law officials and costs of litigation which, depending on the character of the property and its location, may be a comparatively minor item or a very large one;

(*g*) Interest during the period of construction, on money borrowed, or on money invested in the property by its owners;

(*h*) Taxes and insurance during construction; and

(*i*) Contingencies, representing expenditures which cannot be foreseen but which from one cause or another are always of considerable size in the construction or reproduction cost estimate of any great enterprise.

Depreciation and Appreciation.

With a desire to remove the ambiguity and resulting confusion that has attended the use of the term "depreciation" in connection with valuation, the Committee has considered the subject from three standpoints: (1) the cause, *decretion* or loss of service life; (2) the record, *accounting depreciation,* or the money allowance made in bookkeeping to offset accruing loss of service life; and (3) the amount sought, *depreciation of valuation* or *fair depreciation,* the sum which should be deducted from original cost to date or from estimated cost of reproduction new as a step in finding that which the Courts have called "fair value".

Decretion.—This is the fact of loss of service life of a physical property, or property unit, or item, regardless of its effect on value or anything else. It may be due to use, or inadequacy, or obsolescence or accident, either singly or in combination in a given plant or plant unit.

Although, in a well-maintained property, decretion is always present in some degree, yet in some cases this decretion, converted into loss of value, which loss is hereafter called cost of decretion, should not be considered to be a deductible quantity in finding the value of the property—that is, it should not be considered as *depreciation of valuation,* or *fair depreciation.* Unfortunately, this has not been entirely understood.

Whether or not, and to what extent, if any, the loss of value due to existing decretion shall be deducted from original cost to date or cost

of reproduction new, when finding "fair value" for any purpose, is the great troublesome problem of depreciation in valuation.

Accounting Depreciation.—The fundamentals of the methods of accounting for depreciation are that the owner of a public utility is under obligation to the investors in its securities to maintain the integrity of the investment as a continuing property and to furnish suitable service to the public; that the public is under obligation to the owner to pay a fair price for the service rendered, which should cover all operating expenses, a proper allowance for depreciation and a fair return upon the "fair value" of the property; and that the return to the investor and the rates to the consumer should be kept reasonably stable and uniform from year to year and should be fair.

The four accounting methods in use for accounting for depreciation, the *replacement method,* the *straight-line method,* the *compound-interest method* (formerly called by the Committee the "equal-annual-payment method"), and the *sinking-fund method,* yield identical total costs when the whole life of a property unit is considered, and any one of them seeming to be the most convenient may be chosen, provided only that under the circumstances it is legal, safe, and fair.

The replacement method is applicable to those short-lived properties or parts of properties made up of a large number of items, the replacement or retirement of which proceeds after a time with fair regularity and causes no troublesome variations in return or service rates; the straight-line method applies to any property units having more than a year of service life, which are assumed to depreciate uniformly from the beginning to the end of service life; the compound-interest and sinking-fund methods apply to property units the depreciation of which is assumed to progress at the same rate as a sinking fund grows from an annuity, accumulating at compound interest.

In addition to these four methods there is the unit cost method, eminently sound in theory but not readily applicable to accounting purposes, which is based on the conception that the value of a plant unit should be decreased from year to year to such an extent that the cost per unit of output or service, taking into account all annual charges for interest, depreciation, repairs, cost of operating, etc., should be constant during each year of the estimated service life of the unit.

The great discrepancy in the growth of depreciation of long-lived units under the straight-line and compound-interest theories should be carefully noted when determining which method to use.

This report is particularly concerned with depreciation accounting as it relates to valuation, and the determination of the depreciation of valuation. It would seem to be fair in any given case to give the method of accounting used by the utility in setting up depreciation

allowances its proper effect in determining deductible depreciation or the depreciation of valuation.

Depreciation of Valuation Dependent on Accounting Methods and Regulations.—Finding the cost of decretion, here defined as the loss of service life of an item converted into loss of value, is a step in the determination of depreciation, but whether and to what extent, if at all, the estimate thus found shall be treated as depreciation of valuation may be, and very probably will be, dependent, at least in part, on the methods of accounting for depreciation and the character of regulation that have prevailed.

Depreciation of Valuation under Replacement Method.—If by order or sanction of a regulating body, or by long-continued proper custom under no regulation, a property, as for instance a railroad, has been maintained in normal working condition, necessarily less than new in some or all of its parts, by the replacement method, and at any given date is being valued for any public purpose and at that date shows normal condition, all its several parts being in as good condition as could be expected, the accounts showing that those amounts have been expended in renewals that were necessary to keep the property in normal working condition, and the fact appearing that no expenditure reasonably to be expected could put the property in better than the normal condition in which it is found, and that no unusually large expenditure is presently to be necessary for this purpose, then, in spite of the fact that there is an existing decretion in its several parts, there should be found no depreciation of valuation. Under the method of accounting, the public has not paid, and could not pay, for the accrued depreciation, and under this condition its accrued obligation to pay should be considered an asset of the company owner.

If parts of the property are maintained under the replacement method and part by some proper allowance method, then, except as noted below, depreciation of valuation should be found with respect to those parts maintained under the allowance method, but this depreciation of specific physical units will be made good in whole or in part by existing funds or property purchased with allowances, either or both of which will be included in the valuation as they are found.

If in the judgment of the valuing engineer, the replacement method may not be used with propriety for a given property, either because not in accordance with law, or because the method is not adapted to the property, then, whether or not the property has been maintained in the past under this method, the valuing engineer should estimate depreciation of valuation in the amount of the cost of the decretion he finds. There can be no certainty that the property will be properly maintained in the future.

When a comparatively new property, other than a railroad, is to be valued, and it has not been under any regulation that has affected

its accounting methods, the law as laid down in the Knoxville decision would seem to make it necessary to find depreciation of valuation in amount equal to the cost of decretion found for all items, whether or not maintained by the replacement method. The Committee, however, believes that this may work hardship and injustice in some instances, and suggests that in such cases the facts be reported with such recommendations as to equity, as may seem fair to the engineer.

Depreciation of Valuation under Allowance Methods.—If either the straight-line, compound-interest, or sinking-fund method has been used in computing depreciation, and the method of accounting for it has been prescribed by a regulating body or voluntarily followed by a company owner from the beginning, the same theory, so far as it applies to the property in question, should be used for estimating the cost of decretion; and the entire cost so found, lessened by any accumulated depreciation funds, will appear as depreciation of valuation, unless the sinking-fund method of accounting has been used. In the latter case, if the valuation has to do with the reasonableness of the return and the accounting is to go on as before, apparently existing depreciation would not be *depreciation of valuation,* and therefore would not be deductible; but if the valuation has to do with condemnation or purchase, then, as in other cases, the apparently existing depreciation is depreciation of valuation, and the owner should receive the depreciated value of the physical property and the existing fund.

Effect of Regulation on Depreciation of Valuation.—Regulation which determines the method of accounting will in part determine the amount of depreciation of valuation when finding "fair value", because it determines the method by which the public shall pay for the loss of service life. If the regulating body has prescribed the replacement method for the whole period that units have been in existence, then, although depreciation may exist, it is not depreciation of valuation, because, under the method of accounting, the public has not paid, and could not pay, for the accruing depreciation, and is still under obligation to pay for it.

Methods of accounting in force at the present time which make proper provision for the accruing depreciation should not have full weight, if, in previous years during the life of the property units, other methods were in use which did not make provision for such depreciation. The amount of depreciation of valuation in such cases should be equivalent to the accumulated contributions of the public for depreciation allowances under the various methods of accounting which have affected the property unit from time to time. The public is still under obligation to make good that part of the loss of service life not yet paid for, and this obligation should be considered to be as much the property of the company usable to offset accrued depreciation as renewal funds or property actually in existence.

Whether or not this reasoning will stand in any case is for the Court to determine. Valuing engineers and accountants should report what they find as to the actual cost of decretion, and the sums which have been received to offset such cost, under the methods prescribed by the regulating body from time to time. They may give their opinions as to the amount of depreciation that should be deducted from cost to find cost less depreciation as an element in the quantity known as "fair value".

If regulation has not fixed accounting methods, but has limited the earnings, it should be permissible to inquire whether the limited earnings have been sufficient to pay operating expense, depreciation, and fair return. If so, depreciation found should be considered depreciation of valuation to the extent warranted by the accounting methods lawfully or properly followed; if not, a question arises. It is remembered that the duty of the company owner is first to maintain the property "before coming to the question of profit at all", and that it is the duty of the regulating body to see that rates are such as to permit the company owner to earn operating expense, depreciation, and fair return. If the regulating body has made sufficient earnings impossible, is it still the duty of the company owner to maintain the property before paying fair return to its security holders? If it is, depreciation of valuation should be found in the amount of the total cost of decretion or so far as warranted by the accounting methods followed. If not, depreciation found should not be considered depreciation of valuation except to the extent covered by earnings after deducting operating expense and fair return. This is a matter of equity to be determined by a Court. If under regulation the property is a losing venture, it is not included in the class of properties now being considered. In any event the depreciation existing should be found and reported, together with all pertinent facts, that the Court may determine the equities of the case.

Conclusion upon Depreciation.—The valuing engineer should bear in mind that when a company owner has invested a reasonable sum in a property for public service, it is entitled to, but not guaranteed, a fair return on its investment, so long as the money remains in the property, either as property, or funds, or accrued public obligation to pay. Therefore, so long as the company owner keeps a sum equivalent to the total investment at work for the public, either as property serving the public, or funds held in reserve for such property, no policy should be followed in estimating depreciation that will reduce the property to a value less than the investment, or, when using cost of reproduction less depreciation, as a basis of "fair value", to a value less than the cost of reproduction of that part of the property estimated to have been created with company funds, acquired by gift, or in any way not the result of public contributions to cover depreciation.

Appreciation.—Appreciation, largely the result of solidification, seasoning and adaptation, represents the improvement in quality and usefulness of certain parts of the physical properties of a railroad or other public utility property, and it results from the lapse of time, from work not specifically charged to capital account, from maintenance, from use, etc., and covers items, not represented either by the quantities or unit prices, that are determined in connection with a valuation.

There should be no general setting off of appreciation against depreciation, but appreciation should be determined independently from depreciation. Care must be taken that items of labor and expense included in the estimate may not be duplicated in development expense.

Development Expense.

In the production of a normal going property, development expense, almost invariably, is an unavoidable real cost, and is measured by the difference between the amount which the company is entitled to earn in the early years and the amount which it actually does earn. The portion of this expense incurred in tuning up the property and bringing it to its present state of operating efficiency may be included in the cost of construction, and the remainder may be treated as the cost of acquiring the business.

Intangible Value.

The intangible value that pertains to a property and should be given due weight in the ascertainment of "fair value" is the difference between the tangible value—that is to say, proper cost including development expense, less depreciation of valuation—and exchange value, in which is reflected existing and potential dependable income and beneficial results. It embraces going value, in which is merged good will, franchise value, efficiency, favorable business arrangements and design; and it also includes other elements, such as leases, easements, water rights, traffic and operating agreements, strategic location and advantages, and other privileges.

CHAPTER I.

INTRODUCTION.

The Special Committee of seven, appointed by the Board of Direction in September, 1911, to Formulate Principles and Methods for the Valuation of Railroad Property and Other Public Utilities, presented to the Board, early in December, 1913, a Progress Report on the subject of Valuation for the Purpose of Rate-Making.

One member of the Committee was unable to take any part in its deliberations, and consequently did not wish his name attached to the report, which was signed by all the remaining members, consisting of Frederic P. Stearns, *Chairman,* Leonard Metcalf, *Secretary,* Thomas H. Johnson, Alfred Noble, William G. Raymond, and Jonathan P. Snow.

The report was presented at the Annual Meeting of the Society on January 21st, 1914, and, after a brief discussion, the following resolutions were adopted: First, a resolution offered by the Chairman, as follows:

"*Resolved,* That the Progress Report of this Committee, together with all discussion thereon up to September 1st, 1914, or to such later date as the Board of Direction may fix, be referred back to the Committee for further consideration; and that in the meantime the Board of Direction be requested to assign a date for the written and oral discussion on the subject."

Second, a resolution offered by a member:

"That inasmuch as each member of the Society has a copy of that report, that it be not published in the *Proceedings* until the final report of the Committee is issued."

A special meeting for the discussion of the Progress Report of the Committee was held on March 11th, 1914, when the report was discussed by ten members of the Society, and the meeting was adjourned until April 2d, 1914, when it was again discussed in the afternoon and evening by fourteen members of the Society. The total number of written discussions to the present time is sixty-three, covering 340 printed pages of the *Proceedings.*

The Committee suffered a great loss by the death of two of its members: Thomas H. Johnson on April 16th, 1914, and Alfred Noble on April 19th, 1914. To fill the vacancies thus created, and a further vacancy caused by the wish of Henry M. Byllesby not to remain on the Committee, the Board of Direction on May 6th, 1914, appointed to the Committee Charles S. Churchill, William J. Wilgus, and Henry E. Riggs.

Several of those discussing the Progress Report contended that the Committee, in making a report on Valuation for the Purpose of Rate-Making, should have omitted railroads from its discussion, principally on the ground that railroads are competitive, and that their rates are to a large extent interdependent and must be fixed in accordance with these conditions. Some have gone so far as to infer that it was the aim of the Committee to suggest methods for determining individual rates to be included in a tariff, although there was no suggestion of this kind in the Progress Report, which dealt only with returns as a whole; that is, with the aggregate sum to be earned by means of the rates.

The Committee recognizes fully that an engineering valuation of the property of a public utility is only an element in fixing rates, and that this is especially so in the case of railroads and other competitive public utilities. It agrees with the view as to railway rates, expressed by Judge Prouty in his address to the United States Chamber of Commerce on February 11th, 1914, when he says:

"The rates of public utilities are at the present day usually fixed by commissions, both state and federal. It is perhaps the natural inference that when the value of the property has been determined and the rate of return fixed the work of the commission in establishing the charge of the public utility is comparatively easy. It is only necessary to multiply the value by the rate and to allow a charge which will yield that income.

"And this, with some important qualifications, is true as to certain kinds of public utilities. Take, for instance, a water plant or a gas plant. This serves a single community. As a rule it meets no competition in that service. The amount of its business is known or can be forecast with reasonable accuracy. Even matters of depreciation and such like have come to be pretty accurately understood. It is possible, therefore, to fix with some confidence the rates of such a utility when the value of the investment is known.

"With the railroads, however, this is entirely different for the reason that it seldom happens that a single railroad can be considered by itself. The greater part of the business of the railways of the United States is subject to competitive conditions of one sort and another which are largely controlling so that the rates of one are necessarily bound up with those of another. A moment's thought will show the extent to which this is true."

Also:

"* * * the railroads of this country are so bound up together that their rates are largely interdependent. It is impossible to shake a single railroad free from every other and fix its charges upon the basis of a fair return upon its fair value as you would in case of a gas or water plant. The rate established for one, of necessity, influences and frequently absolutely determines the rate of all, a fact which must never be forgotten in discussing this subject."

In the same speech, however, he adds:

"While, however, I wish to make it perfectly plain that the problem of establishing railway rates will not be solved by this valuation, I desire to say with even greater emphasis that the problem will be enormously simplified. It can be known with certainty whether the general level of rates is or is not too high, and in establishing the charges to be observed by a single carrier, even in fixing the rate upon a single commodity, it will be of much benefit to know the value of the property involved. Every railroad commissioner will join with me in saying that here is the only solid foundation upon which he can stand; that the determination of these values is indispensable to the just and intelligent administration of his work.

"While this valuation will be of incidental benefit to the investor, while it is essential to the work of the rate-making tribunal, it seems to me that its greatest immediate value is political. The state of the public mind towards our railways is such that this information is absolutely necessary."

Whether or not it is proper to base rates on the so-called "value" of the property is no concern of the Committee. The fact is that in many instances, including cases of railroad rates, the reasonableness of rates has been so determined, and, in making their decisions in disputed cases, the Courts have quite uniformly held that a public utility corporation is entitled to earn a "fair return" on the "fair value" of its property used in the public service.

Whenever the determination of the reasonableness of rates is reached in this way, it is necessary to value the property, and to value it in such a way that justice shall be done to the corporation and to the public. As this use of valuation of public utility properties was and now is prominently in the public mind, it seemed desirable to consider first the methods of valuation for this purpose. Hence, the Progress Report was limited to this field.

There seemed to be a failure to understand completely what the Committee proposed in its Progress Report, and there were those who contended that there could be but one value for any plant operated for gain, no matter what the purpose of the valuation. Further consideration by the Committee leads it to believe that much, if not all, of the misunderstanding is due to the unfortunate multiple meaning of the word value. In its own discussion the Committee has had difficulty in reaching conclusions, due to this cause. In its Progress Report the Committee adopted the current usage of the word value in accordance with the precedent set by Courts, authors, and legislatures. One of the most prominent of the authors of works on valuation has recognized the difficulties attached to the multiple meanings in the current usage of the word, but dismisses the subject with the statement that the substitution of other terms is not without difficulties.

After careful consideration, the Committee has concluded that the general use of a modified terminology would go far toward lessening the opportunity for misunderstanding, and therefore suggests that the use of the word "value" be restricted in general to its ordinary significance as defined by the Standard Dictionary: "The desirability or worth of a thing as compared with the desirability of something else," and has recognized the propriety of the use of other terms than "value" to express the bases on which purchase, rate-making, or the determination of the reasonableness of rates, capitalization, or taxation must rest. (See Glossary under VALUE.)

The Committee adopts this terminology except when it is necessary to use the terms "value" and "fair value" with the meanings attached to them by the Courts, in which case they will be enclosed in quotation marks.

The Committee has attempted in this report to cover the whole subject assigned to it, which requires a very extended report, even when it is limited to the general features of the subject. To cover all questions which arise in connection with valuation would be impracticable in a report by a committee of this Society.

The following preliminary statements may assist to an understanding of the discussions which follow:

1.—Although the report deals with the principles and methods of valuation, it lays stress on the fundamental principles. General methods are indicated, but little attention is given to detailed methods which necessarily vary under different circumstances.

2.—The discussions, unless otherwise stated, are confined to normal properties which are neither overbuilt, inadequate, nor improperly located.

3.—It is the view of the Committee that valuation should have its foundation in fact, and that the results obtained should be those which will be fair to the parties affected.

4.—The interdependence of methods of valuation and of regulation of public service properties must be recognized. Different methods of regulation may be adopted, which, taken in connection with the appropriate methods of valuation and consistently followed, will be fair to the parties affected. It is not, however, immaterial which of the fair methods are adopted, because one may be more closely in accordance with the law as defined by the Courts than another, and hence may be more capable of being carried out effectively.

5.—Where there may be two or more practicable and lawful methods of procedure, it is still desirable to aim toward uniformity and to select the most approved methods for general use, as this will tend toward simplicity and the avoidance of that confusion of thought and action which occurs when such uniformity does not exist. The methods

recommended by the Committee are those which, all things considered, are in its judgment most desirable for general use.

6.—The various elements which enter into the valuation of a property are frequently interdependent, and equitable results can be obtained only by a recognition of this fact.

7.—As indicated by the preceding paragraphs, a sharp distinction should be made between what may be done now under present laws and what may be done under future legislation and continuous commission control. The recommendations applying to present conditions should not be confused with those which can become practicable only with such legislation and control.

8.—The Committee has devoted the major portion of this report to a consideration of principles and methods which should govern in the valuation of the physical property plus development expense, or, in other words, what it has cost or would cost to create the property and to create the business. The valuation so determined does not cover all the elements of value which should receive consideration from the Courts and the various regulating bodies; therefore, a portion of the report is devoted to a discussion of intangible values and to other matters which may in many cases have an important bearing in the final determination of "value".

9.—Until the science of valuation and the adoption of sound accounting methods have advanced much further than at the present time, it is probable that different methods will have to be used in different cases to produce fair results, and special cases will always continue to arise which require methods adapted to them. The aim, however, should be toward uniformity.

10.—In questions relating to valuation, interest is an important item which should always be considered, and when interest is mentioned in this report it means interest compounded annually.

The valuation of public utilities has obtained special importance in recent years, and most of the literature relating to this subject has been issued within the last sixteen years. The principles and methods involved are now being carefully considered by many public service commissions and the Courts, as well as by engineers, lawyers, and committees of various societies having to do with public utilities. The subject is still in the developmental stage. Until recently, there were no engineering or legal works devoted exclusively to the valuation of public service properties, but since 1911 several important and valuable books on this subject have been published.*

* See "Valuation of Public Service Corporations," by Robert H. Whitten, New York, 1912 and 1914, a comprehensive work in two volumes in legal form, which contains a discussion of the various principles involved in the valuation of the property of public service corporations, and is replete with extended quotations from the opinions of the Courts; also, the following: "Valuation of Public Utility Properties," New York, 1912, and "Value for Rate-Making," New York, 1916, by Henry Floy; "Public

The Committee, in preparing this report, has carefully considered the views expressed in these recent works and in the many written discussions on the Progress Report.

Utilities, Their Cost New and Depreciation," New York, 1913, and "Public Utilities, Their Fair Present Value and Return," New York, 1915, by Hammond V. Hayes; "Principles of Depreciation," New York, 1915, by Earl A. Saliers; and "Public Utility Rates," New York, 1916, by Harry Barker.

See also Bibliography to July 16th, 1913, containing 61 pages, prepared in the Library of the American Society of Civil Engineers, under the direction of the Secretary, at the request of the Committee, *Transactions*, Am. Soc. C. E., Vol. LXXVI, December, 1913, p. 2133. This bibliography, together with an additional 72 pages, bringing it up to December 23d, 1915, has been published by the American Electric Railway Association.

CHAPTER II.

FUNDAMENTAL PRINCIPLES OF VALUATION.

A fundamental principle of primary importance is that a valuation shall be made so that it will be fair to all parties affected.

So long as it is desired that public service corporations and not government shall provide the utilities required by the public, it will be necessary, in the exercise of public regulation, to use principles and methods of valuation which, when properly applied, will attract the capital necessary for building new properties, for extending old ones, and for rendering satisfactory service.

Under normal conditions, the investor is induced to create new property by the prospect of obtaining a fair return on the amount of his investment*, and such investment is a fair basis for valuing such new property, but is not necessarily the "value".

Not many years ago, it was very common to value public service property by capitalizing the net earnings, and though the sum thus obtained may be a proper element for consideration under some conditions in the valuation of a property, it is manifestly not a proper basis on which to predicate reasonable returns.

The fundamental principles involved in the valuation of a new property are necessarily different in some respects from those involved in the valuation of an old one, because a new property, which has not begun to earn or to have the value of its parts affected by appreciation or depreciation, stands in a different position from an old one which has been in operation and earning for many years, and has many parts which may have appreciated or depreciated in value. Moreover, in the case of an old property, valuable property or rights may have been acquired or lost, without the payment or receipt of money therefor, as a result of time or through the failure of the public or the owner of the property to assert their authority. Property or rights so gained or lost affect the valuation materially, and a failure to recognize them would be inequitable to the present owners of the property and to the public.

A more extended statement will be given of the fundamental principles involved in the valuation for various purposes of these three classes of property, namely:

New properties;

Old properties under commission control from their inception;

Old properties which have not been subject to continuous regulation.

* The word, as used here, is intended to mean the reasonable and proper investment for creating the property, and not a reckless or improvident investment.

Valuation of New Properties.

If a new property, just ready for operation, is to be valued in connection with the determination of fair return, then, because working capital will be required, unfinished work must be completed, and development expense incurred, a reasonable and proper basis for what the Courts have called "fair value" is the actual investment in the property devoted to public use plus an estimated sum for working capital, final completion, and development expense, the word "investment" being defined, as in this report, to include not only the outlay of money but also the money value of services and other considerations involved. Such a basis is the most rational one to use in this connection, because a fair return on such a sum, would be a fair return on the investment, and this is what is needed to attract capital to such properties.

If the same property, just ready for operation, is to be valued for public purchase or in connection with condemnation proceedings by a public body, such as a municipality, the public purchase base of a successfully constructed property should be the actual cost, including the money value of services and other considerations involved.

A valuation of the same new property for capitalization should be determined in the same way as the return base, except that all parts of the property, whether devoted to the public use or not, should be included.

A valuation for taxation of a public service property, whether new or old, must accord with the laws of the State in which it is situated, whether or not the laws are equitable, and, where not governed by such laws, should, to make it equitable, be fixed at a sum consistent with the valuation of other property for taxation.

Valuation of Old Properties Under Commission Control from Their Inception.

Some of the basic principles involved in the valuation of old properties can be discussed best by taking a hypothetical case of a normal, well-established property, operating without competition and under commission control as to rates, methods of accounting, etc., from its inception. It is assumed that such a commission would have acted throughout the life of the property in accordance with the basic principles which should govern in such a case. Some of these are:

(*a*) The owner of the property is entitled to reimbursement through the earnings for all current expenses for operating the property and maintaining in good condition the units of which it is composed, including the amounts paid for taxes.

(*b*) The owner of the property is entitled to an allowance sufficient to provide for the net depreciation in the value of all units of physical

property, whether resulting from decay, wear and tear, or other causes, the amount of such depreciation allowance to be sufficient to pay for all such property units by the time they cease to have value.

(*c*) The owner of the property is entitled to a fair return on a "fair value" of the property, and such return should include, not only the ordinary rate of interest for the use of well-secured capital, but in addition a profit commensurate with the risks incidental to the investment.

(*d*) If, during the early years of operation, before the property is tuned up and the business developed, it is impracticable, as generally is the case, to earn the current expenses (*a*), a suitable depreciation allowance (*b*), and a fair return (*c*), the owner of the property should be compensated in some way for the loss during the early years.

The first of these basic principles (*a*) is universally recognized.

The second (*b*) does not define whether the depreciation allowance shall be annual or otherwise, but the decision of the United States Supreme Court in the Knoxville case supports the equitable principle that the allowance for depreciation should be earned annually and should be of sufficient size to keep the investment intact. A portion of the decision is as follows:

"* * * the company is entitled to earn a sufficient sum annually to provide, not only for current repairs, but for making good the depreciation and replacing the parts of the property when they come to the end of their life. The company is not bound to see its property gradually waste, without making provision out of earnings for its replacement. It is entitled to see that from earnings the value of the property invested is kept unimpaired, so that, at the end of any given term of years, the original investment remains as it was at the beginning."

The third basic principle (*c*) is applicable to normal properties and especially to those not subject to competition, and is in accordance with the general view which has been expressed by the United States Supreme Court for many years, namely, that the owner of a property is entitled to a fair return on the "fair value" of the property.

The fourth basic principle (*d*) can be applied in practice either by increasing the "fair value" of the property by the sum of the deficiency of earnings in the early years, with interest, or by allowing higher rates of return in subsequent years to offset the early deficiency. This subject is further discussed under the head of "Development Expense".

Valuation of Old Properties Which have not been Subject to Continuous Regulation.

The basic principles (*a*), (*b*), (*c*), and (*d*), given on the preceding pages, are important in connection with old properties not subject to

continuous regulation, but there is another basic principle already referred to, relating to such property, which in many cases makes the sum representing the sacrifice by the owner an inequitable basis of "fair value". This is:

(*e*) Valuable property or rights may have been acquired or lost by the owner of a public service property, as a result of time or through the failure of the public or the owner to assert their authority.

A failure to recognize such property or rights in a valuation at the present time would be unfair to the present owner or owners of the property or to the public.

The basic principle (*e*) may apply to many features of an old public service property, but two illustrations will indicate what is intended.

Years ago, in the absence of restrictions on rates, a public service corporation may have had very large earnings, and, after paying out adequate dividends, may have had a large surplus, which it expended in increasing and improving its property. Such action was then regarded as commendable. If the public for many years has neglected to exercise its right to regulate rates, it is only just and equitable that the increase and improvement in the property thus created should be recognized, and that the present-day owners should not be penalized for this neglect.

As an instance of a loss by the owner of an old property, he may long ago have neglected to charge rates sufficient to maintain his investment intact, with a resulting diminution in the value of his property. The public, many years afterward, could not be expected to make good the deficiency of former years. This is the view taken by the United States Supreme Court in the Knoxville case, where it states:

"If, however, a company failed to perform this plain duty and to exact sufficient returns to keep the investment unimpaired, whether this is the result of unwarranted dividends upon over issues of securities, or of omission to exact proper prices for the output, the fault is its own. When, therefore, a public regulation of its prices comes under question, the true value of the property then employed for the purpose of earning a return cannot be enhanced by a consideration of the errors in management which have been committed in the past."

The foregoing discussion is based chiefly on grounds of equity, but it is also true that the Courts recognize as a general rule that it is the present value of property which should be used, and not its cost.

Where the sum representing the sacrifice by the owner cannot be accepted as a basis for valuation, it is necessary to find other bases, and a discussion of these and the fundamental principles in each case will be found in the chapters headed "Original Cost to Date" and "Cost of Reproduction".

CHAPTER III.

PHYSICAL PROPERTY TO BE INCLUDED IN VALUATION.

The physical property to be included in valuation varies in different cases with the use to which the valuation is to be put and the law governing the case. It is desirable to cover the entire property, placing different classes in separate schedules, so that full information may be available to the Court or commission having the final decision as to the property to be included.

The most important questions which arise in making schedules are those which ask whether, and to what extent, property not devoted to the public use, property retired from service, temporary works, donated property, leased property, working capital, and securities owned shall be included. Is it proper to exclude a part of the total value of works on the ground that they have been built of a size in excess of that required for the service? Is the title to a property a controlling feature in determining whether or not it shall be included in the schedule? All these questions will be discussed in this chapter.

Used and Unused Property.

It is the well-established rule of the Courts that when a valuation is made for the purpose of rate regulation, only that property is to be included which is devoted to the public use or is used for the public convenience. In a valuation for capitalization, it is obvious that all the property should be included. In the case of condemnation of a property which is taken over by the public authorities, all the property condemned would be subject to valuation, and according to law every element of its value should be included.

The opinion of the United States Supreme Court with regard to the property to be included in valuation for rate regulation is shown by the following brief quotations from decisions, covering the period from 1898 to 1913:

"The fair value of the property being used by it for the convenience of the public." "What the company is entitled to ask is a fair return on the value of that which it employs for the public convenience." (Smyth *v.* Ames, 169 U. S., 466, March 7th, 1898, page 546.)

"The then value of the property actually used for the purpose of supplying water." (Stanislaus County *v.* San Joaquin and King's River Canal and Irrigation Co., 192 U. S., 201, January 18th, 1904, page 213.)

"The fair value of its property devoted to the public use." "The value of its property actually used for the public." (Willcox *v.* Consolidated Gas Co., 212 U. S., 19, January 4th, 1909, page 50.)

"The basis of calculation is the fair value of the property used for the convenience of the public." (Minnesota Rate Cases: Simpson *et al. v.* Shepard; Same *v.* Kennedy; Same *v.* Shillaber, 230 U. S., 352, June 9th, 1913, page 434.)

In view of these decisions, it is clear that the law does not provide for the inclusion of unused property in a valuation for the purpose of rate-regulation, but there must necessarily be much latitude in the determination of the property to be classed as "used" and "unused". Although a few of the discussions by the Courts indicate a tendency toward the exclusion of all property not "actually used", in cases of doubt both Courts and commissions have in practice favored including rather than excluding property.

Clearly it is a narrow construction of the law to claim, as has sometimes been done, that all property not actively in use at the time of the valuation should be excluded. A broader policy is desirable, both in the interests of the owner of the property and of the public. The property considered to be devoted to the public use, and therefore to be valued, should include, not only that in active use in the every-day operations, but that which is properly and reasonably held in reserve to insure the sufficiency and continuity of the service.

In general, it is not proper to apply any arbitrary rules excluding property temporarily out of use. Railroad equipment may be stored during periods of depression; snow-plows and flangers are in use only a small part of the time, and thousands of other cases might be cited of property not actually in service at the time of the appraisal, but useful, and essential as a part of the equipment of the property. In like manner, the duplicate pumps in a water-works, and reserve power units of all kinds, are essential to the proper operation of the property and necessary in times of emergency, although not actually used any considerable proportion of the time. No rule of valuation which excluded such property could be sustained by sound reasoning, as the very life of the service depends to a large extent on sufficient reserve to tide over emergency or peak of business. Such property should be included in regular schedules.

Recognizing that the erection of manufactories and other buildings, the opening of new streets, the laying out of parks, and the making of other customary improvements in the neighborhood of public utilities, as well as increases in the value of adjoining property, will make the future acquisition of lands for the expanding needs of such public utilities difficult and expensive, if not impossible, it has been customary for public utilities to exercise foresight in the purchase of surplus land at crucial points, and in the opinion of the Committee it is in the interest of both the owner of the property and of the public that such land purchased in good faith and held in reserve for future use

should be included in the valuation, even though portions are for a time not in active use.

The following extract from a decision of the New York Public Service Commission for the Second District, in a rate case, is quoted as an instance of what appears to be a rational treatment of the problem of determining what portion of certain lands should be included as used property (Buffalo Gas Co. *v.* City of Buffalo, 3 P. S. C., N. Y., 2 D., 553, February 4th, 1913, pages 578-579):

"All of the land at the Genesee Street plant should be treated as in the public service. The contention of the city that a certain parcel next to the canal is not in the public service should not be allowed. It is not in fact used at this moment, but it is directly adjacent to the generating plant of the company, and with any growth in the business would undoubtedly be needed; and justice and fair dealing do not allow for a moment the quantity of land to be scaled down to the lowest point possible under present circumstances. The company is fairly entitled to a reserve of land of this character at this location * * *.

"At East Ferry Street there are 5½ acres of land. The only use it has in the public service is that of a holder station; 5½ acres for this purpose is not shown by any evidence in the case to be warranted. As a matter of fact, but a portion of this land is used for that purpose; and there is no evidence showing that more will be required for public use within a reasonable time in the future or in fact at any time. An allowance of one-half of this land as being in the public service is liberal to the company.

"At Forest Avenue there are two parcels of land separated by the tracks of the New York Central railroad. The smaller tract is rented for and used as a coal yard, and has no connection whatever with the public service. A considerable portion of the larger tract next to the water is not in fact used in the public service at the present time. The holder and engine-house are situated at one end of the lot, and some small cheap buildings at the other end which are not used for gas purposes. Owing to connection with the street, location of piping, etc., it is believed that it may not be unreasonable to treat the whole tract as being in the public service."

Although the Committee believes that the law relating to used and unused property should be interpreted liberally, as above indicated, it recognizes that, were it the policy to value all land which the owners of a public utility might purchase, on the ground that it would be needed for future use, such policy might lead to speculation in land to be held for a time at the expense of the public and subsequently sold at a profit, without having served it.

A very narrow view is sometimes taken on the area of land to be included in a valuation for rate regulation, as for instance in the case of a recent railroad rate investigation where it was claimed that there should be excluded from the valuation (*a*) all lands owned in terminals and large cities not actually in present use and covered with tracks,

buildings, or company materials, and (*b*) all portions of the right of way not actually included within the slopes of cuts or fills.

The width of a railroad right of way is determined by the depth of excavations or embankments, the necessity of securing materials for forming the banks, the need of providing room for the operations of contractors, as well as by consideration of the necessities of operation.

Another case called to the attention of the Committee is an objection to the inclusion of lots in a city making a right of way 225 ft. in width, while the present use of the property is to support a double-track bridge 65 ft. from the top of rail to ground surface, the present structure covering the central 50 ft. In this case the entire strip was amply justified on three grounds: (1) that it was all needed for the construction and maintenance of the present large timber structures and that any less would give insufficient room; (2) that the land owned was barely sufficient to support an earth embankment if that form of permanent structure should finally be determined upon; and (3) that the land was necessary for the protection of its existing structure from encroachment of buildings which would create a fire hazard.

It is the opinion of the Committee that, in a valuation for the purpose of rate regulation, there should be included, not only the lands actively in use and covered by the construction, but such additional area as is necessary to permit the economical construction of the works, to safeguard the property and protect it from such encroachments as might interfere with its operation or create an extra hazard and to provide for permanent in place of temporary construction. Good business judgment in the acquisition of land frequently requires the purchase of a whole parcel of land where the greater part of the parcel is required for the purposes of the utility. All the land thus acquired should be included in a valuation.

If a 100-ft. strip of land constituted a proper width for constructing a certain single-track railroad and permitted the construction operations to be handled economically and properly, no part of that right of way should be excluded on the ground that the embankments only occupied space equivalent to 41 ft. average width.

Equally important is the question of inclusion or exclusion in valuation for rate regulation of lands held in reserve for future extensions.

In very many, if not most, cases where this question is raised it will be found that a tract of land has been acquired sufficient to provide a suitable site for the proposed plant and to provide for such extensions as are reasonably likely to be made in the near future. It is poor business judgment to build a gas plant, an

electric plant, a railroad shop yard, or other like property, on land just sufficient to hold the buildings and other improvements immediately necessary, without reasonable provision for expansion. The history of all properties that have been built in response to reasonable demand for their services has been a history of growth, development, and expansion. Good business policy would provide for this expansion so far as it can be reasonably anticipated, and all property should be included which is not in excess of a reasonable amount for development of the business over a reasonable period of time in advance of the valuation. Such a determination is not a matter of pure speculation. An investigation into the history of the growth, expansion of business, and expansion of physical property of the company, together with a consideration of existing conditions at each site, should easily determine the reasonableness or unreasonableness of the inclusion.

A very mooted question as to the inclusion of land is found in the case of a company which has bought such property, disconnected from existing works, for future use and development. Often many large holdings are acquired some time in advance of development, at prices much lower than would prevail if bought when the need was pressing and the transaction forced. The general policy of the Courts appears to have been to exclude such holdings in cases relating to rate regulation, unless it was apparent that the property was to be used in the near future.

While the rulings of the Court in regard to the exclusion of such reserve properties, or properties which have not yet come into use, may be justified on the ground that the company may defer its purchase unless it is willing to take the risk that the advance in value will furnish sufficient compensation for holding the property, and while it is true that there are cases in which to include all such properties would lead to abuse, the Committee is of the opinion that there are other cases in which it is very desirable, in the interests of economy and efficiency, that corporations should anticipate the future needs of the service, and that the reserve properties held to meet such future needs should be included in valuation. Instances might be: a distributing reservoir site on the top of a hill, purchased before the hill property is sub-divided into streets and house lots; a dock site on a deep-water harbor for the future use of a railroad; and water rights acquired when they can be obtained at small cost, lest by reason of later developments the company may have to pay heavy consequential damages or substantially monopoly prices for them, or even be unable to obtain them if they have been appropriated by others, the last instance being particularly applicable to the semi-arid and arid regions in the Western and Southwestern United States.

The foregoing views may be less applicable when the valuation of the land purchased for future use is based on the reproduction cost, than when based on original cost, because, if the public is required to pay an increased rate on account of the inclusion of land which is not used, and the increase in the value of land as shown by the difference between reproduction and original cost inures to the company, an inequitable result may be reached.

In the actual work of making a valuation, there are many cases in which there would be no difficulty in reaching a decision as to whether or not a given property should be classed as a part of that used for the convenience of the public. For instance, property used for the commercial mining of coal, although owned by a railroad company, obviously should not be included in a valuation of the property of that company devoted to the public use, but there are other cases in which it is more difficult to decide. Among these may be mentioned office buildings owned by public service companies, which are rented wholly or in part to the public, property temporarily leased to others, pending its active use in the service of the public, and hotels and parks owned by railroad companies. Matters of this kind generally have to be settled by a consideration of the specific cases, on the basis that properties which are a part of the operating plant of the company or are being used for the convenience or benefit of the public should be included. When so included, the income arising from the operation of these properties should be duly credited.

Retired or Discarded Property.

Closely allied with, and in fact a part of, the unused property is that which has been used by a public service company and has been retired or discarded prior to the time of the valuation.

Under the rulings of the Supreme Court, and in accordance with many rulings of other Courts and of commissions, such property would be excluded from a valuation. There are cases in which such rulings seem to be opposed to fair dealing to the owner of the property abandoned. Instances of this kind may be found where a telephone or electric light company is suddenly ordered to remove all overhead wires and place them underground, before the company has had an opportunity to charge off the cost of the overhead wires.

Under public regulation at the present time, due account is sometimes taken of that part of the cost of the discarded property which has not been charged off, and the public service company, instead of being required to charge the remaining cost of the discarded property at once to operating expense, is permitted to charge it to a suspense account for the purpose of distributing the loss to the operating expense of a succeeding period of years.

The Interstate Commerce Commission, in its uniform system of accounts for telephone companies, referring to such losses, states ("Uniform System of Accounts for Telephone Companies", First Issue, effective January 1st, 1913, page 68):

"Losses of this sort may be due to the requirement by lawful authority or public necessity of improvements involving the abandonment of a considerable portion of plant and equipment before it has attained its normal life in service, or to an extraordinary casualty entirely unforeseen and unprovided for. The original cost of the property so abandoned or destroyed should be credited to the fixed capital accounts in which it was carried, and such portion of the cost as may be authorized by the Commission may be charged to the suspense account, the remainder of the cost, less any salvage, being charged out as elsewhere provided in case of retirements of property. The suspense account so raised should be credited and account No. 609, 'Extraordinary Depreciation', debited monthly with such an amount as will, through its regular application, amortize the amount of the loss at the end of the period designated."

Similar provisions occur in the systems of accounts for other carriers.

In a few instances public service commissions have permitted the inclusion of discarded property in the valuation for a term of years. As an instance, a decision by the California Railroad Commission in a rate case is quoted (Solari *v.* Tuolumne County Electric Power and Light Co., 3 Cal. R. C. R., July 29th, 1913, pages 1051-1052). The valuation was that of an electric plant.

"It seems fair, however, to permit the defendant during each of the ensuing ten years to collect rates high enough to permit it to charge off such sum on its books that by the end of the ten years the principal so charged off, together with the interest, shall have amounted to the entire value of the line at the present time. I believe that it will be very liberal on the part of the rate-fixing authorities to permit this to be done."

Where plant units have been worn out in service and definitely abandoned, or by reason of the reconstruction of property, parts have been abandoned, and are not only unused but not likely again to be used, such units and parts should appear either in a separate schedule of retired or discarded property, or such full statement of the conditions should be made as will definitely fix the status of the units or parts.

There is another class of property which, even though out of use, should not be excluded from a valuation on the ground that it is retired or discarded property. This includes the temporary works required for the construction of the permanent items of physical property, as, for instance, the temporary falseworks used to support a bridge during its construction, the coffer-dam required to keep the

site of a dam or a bridge pier free from water, or material and labor of preliminary track surfacing required for the protection of rails from injury under construction traffic prior to the commencement and completion of permanent ballasting. Although such temporary works are discarded after the completion of the permanent works, their cost is clearly a part of the cost of the permanent works, and is properly includible when determining unit prices.

Similarly, the cost of temporary works which are necessarily built in connection with the construction of permanent works, as, for instance, the cost of a pile bridge built in advance of the completion of a permanent bridge for the purpose of expeditious and economic construction, should be included in the cost of the permanent bridge.

In accordance with the same general principle, it seems proper to include in a valuation the cost of temporary works built to furnish service to the public at an earlier date than it could be furnished by the permanent works. For instance, a temporary water-works pumping station is sometimes provided for use during the construction of a permanent pumping station. Such works may be advantageous to the public as well as to the owner of the public utility, and their cost may be regarded as a part of the cost of construction, although due allowance should be made for receipts from operation.

Excessive Size or Capacity.

Somewhat allied to the question of used and unused property is that of excessive size or capacity. It is sometimes claimed that the public should not be required to pay rates on works having a size or capacity in excess of that required for the purposes of the community served. Rulings on this feature of valuation have been made by the Courts and commissions in several instances.

Judge Savage of the Maine Supreme Court used this illustration (Brunswick and Topsham Water District *v.* Maine Water Co., 99 Maine, 371, Dec. 14th, 1904):

"Suppose that a five-hundred horse-power engine was used for pumping when a one-hundred horse-power engine would do as well. As property to be fairly valued, the larger engine might be more valuable than the smaller one, yet it could not be said that it would be reasonable to compel the public to pay rates based upon the value of the unnecessarily expensive engine."

Judge Pitney, of the New Jersey Chancery Court, diminished the valuation of a water plant by nearly $130 000 out of a total of $1 400 000, on the ground that a 36-in. main was used where a 30-in. main would have been sufficient, and because a dam was made of excessive width for present purposes in order that it might be raised subsequently. (Long Branch Commission *v.* Tintern Manor Water Co., 70 N. J. Eq., 71, November, 1905.)

In most of the commission decisions stress is laid on the necessity of providing adequately for future needs in order that satisfactory service may be furnished continuously, and it is stated that it is not desirable in the general interest of consumers to discourage a reasonably liberal provision for the future.

The Canada Board of Railway Commissioners, in the application of the City of Montreal for the reduction of telephone rates, made the following statement, October 28th, 1912:

"There is no question that preparation for future needs is one of the incidents of the proper management of a public utility corporation. If it is to allow demands for service to pile up and then make an expansion only after the urgency is sufficiently great, the public will be subjected to the disadvantage of delay in obtaining service, and at the same time the piecemeal method of construction this will necessitate will undoubtedly add to the cost of the plant. A comprehensive system of preparation for future needs must be followed if there is to be proper expansion. Undoubtedly this will normally lessen the cost of construction to the company. It is also of advantage to the public using telephone service because it gives a decreased basis on which earnings are to be obtained."

In the case of the Des Moines Water Co. *v.* City of Des Moines, the Master, in his report of September 16th, 1910, says:

"* * * the company has the right to anticipate the growth of its business and to be allowed a proper return on a plant of sufficient capacity for such growth."

The Massachusetts Gas and Electric Light Commission, in the Haverhill gas rate case, decided December 31st, 1912, made this statement:

"The Board can not, in the general interest of consumers, discourage the reasonably liberal provision for the future which this company seems to have made."

It is the judgment of the Committee that there should not be a reduction in the valuation on account of excessive size or capacity, except when the excess is so great as to be clearly unreasonable and is the result of not using proper foresight. The opinions of engineers and others engaged in building public service property differ materially as to the length of time for which it is economical and proper to make provision under the different circumstances, and property built with reasonable foresight should not have its value diminished because expectations have not been fulfilled by subsequent developments.

If the opposite course is pursued, corporations may be deterred from making wise provision for the future, thereby increasing the hazard of the investor, probably increasing the ultimate cost by reason of premature retirement, duplication, or piecemeal construction, and hence ultimately increasing the cost of service to the public.

Donated Property.

The Committee believes that land or other property which has been voluntarily donated by the public or by private parties to a railroad or other public utility should be included when determining the reproduction cost, on the same basis as land and property otherwise acquired.

It is especially true that property donated many years ago should be so included, because the company which has held such property for a long term of years, with the acquiescence of the public, has acquired full rights therein, even if it did not have them originally. Such donations were generally made because companies could not profitably furnish a demanded service without such assistance. The donations were necessary to prevent the proposed project from becoming a "losing venture".

There are some instances in which a company has the title to a given property which should not be included in the reproduction cost, and, conversely, other instances in which a company has no title to a given property which should be included in such cost. This subject is discussed on subsequent pages of this chapter, under the heading "Title to Property not Conclusive".

Leased Property.

In the case of leased property, either the property itself or the lease should be valued, as circumstances may dictate.

When a valuation is made for the purpose of rate regulation, it is usually customary and proper to include in it property owned or leased for long-term periods and used for the convenience of the public, although the valuations of the owned and leased property should be kept separate for the information of the Court or commission. Short-term leases may be given a positive or negative value, according to whether they represent prospective profit or loss.

When a valuation is made for the purpose of capitalization, property leased to and not owned by the corporation whose property is being valued should be excluded, although there may be exceptions to this rule in the case of leases which for any reason are practically equivalent to ownership.

When a valuation is made as a basis for selling the property of the operating company, the value assigned to leaseholds should depend on the duration, character of control, and advantages or disadvantages accruing to the lessee by reason thereof.

The foregoing statements are chiefly applicable to leases made between public service corporations, and may not apply to the many instances in which minor leases are made between a public service corporation and private parties.

Title to Property Not Conclusive.

When a valuation of a public service property is made, it is frequently necessary, in order to be fair to all parties interested, to depart from the usual rules relating to ordinary private property, and to include in the valuation more or less property situated on land to which the owner of the public utility has no title, and, on the other hand, to exclude property situated on land to which he has a title.

The proper general rule, subject to exceptions, appears to be that the owner of a public utility is entitled to have included in the valuation of his property all those works for which he has been required by law or necessity to pay the cost, including in this class also property voluntarily donated, and that there should be excluded all those works for which other public service companies, the public or the users have been required by law or necessity to pay the cost.

For instance, it would be unfair to require a railroad company, as a part of the work of abolishing grade crossings, to raise highways in order to furnish suitable approaches to an overhead bridge, and to exclude the embankments thus formed from a schedule of the company property to be valued, on the ground that the company did not have the title to the land on which they were located, or for the same reason to exclude from the schedule in the valuation of street railway property the pavements which the company may have been required to lay in connection with its tracks.

Similarly, in valuing a water or gas property, it would be unfair to include in the schedule those service pipes from the mains to the street line that have been paid for and are maintained by the consumers of the water or gas, unless contractual obligations shall determine otherwise.

The pavements over the mains laid by gas and water companies furnish further instances of property to which the companies have no title, but it is generally conceded that the companies are entitled to have included in an estimated cost of reproduction the cost of removing and replacing such pavements as were actually removed and replaced by them when laying the mains.

The propriety of the inclusion or exclusion of the cost of removing and replacing the paving over mains not actually removed and replaced, is discussed in the chapter on reproduction cost. There is, however, a difference of opinion as to whether the cost of removing and replacing pavements laid at the expense of the public after the mains were in place should be so included. In accordance with the rule above stated, they should not be included, though upon a strict interpretation of reproduction under present conditions, the pavements would have to be cut, and if they can be shown in fact to increase the value of the pipe system, and the property as a whole.

this increase in value may be considered. In the most recent decision of the U. S. Supreme Court, however, the Court has held that they did not in fact increase value in the case under review.

The more recent and important Court and commission decisions exclude from "fair value" those pavements and other items laid or constructed without expense to the company.

In 1907, the decision of District Judge Hough, of the Federal Court, in the Consolidated Gas Co. case (Consolidated Gas Co. *v.* City of New York, 157 Fed., 849, December 20th, 1907), included in "fair value" the increased construction expense of pipe systems over which pavements had been placed at the expense of the city. On appeal, the United States Supreme Court (Willcox *v.* Consolidated Gas Co., 212 U. S., 19, January 4th, 1909) did not take up the question of the inclusion of pavements, but concurred with the Court below to this extent:

"And we concur with the court below in holding that the value of the property is to be determined as of the time when the inquiry is made regarding the rates. If the property, which legally enters into the consideration of the question of rates, has increased in value since it was acquired, the company is entitled to the benefit of such increase. This is, at any rate, the general rule."

In 1912, District Judge McPherson, of the Federal Court, in the case of the Des Moines Gas Co. *v.* City of Des Moines (199 Fed., 204, August 21st, 1912), supported the exclusion by the Master of the reproduction cost of pavements laid at the expense of the city, thus taking a position diametrically opposite that taken by Judge Hough.

Upon appeal, this case went to the United States Supreme Court (238 U. S., 171, June 14th, 1915), which supported the attitude of the Master and of Judge McPherson regarding the exclusion of pavements laid at the expense of the city. Mr. Justice Day says:

"As to the item of $140 000, which, it is contended, should be added to the valuation, because of the fact that the Master valued the property on the basis of the cost of reproduction new, less depreciation, and it would be necessary in such reproduction to take up and replace pavements on streets which were unpaved when the gas mains were laid, in order to replace the mains, we are of opinion that the court below correctly disposed of this question. These pavements were already in place. It may be conceded that they would require removal at the time when it became necessary to reproduce the plants in this respect. The Master reached the conclusion that the life of the mains would not be enhanced by the necessity of removing the pavements, and that the Company had no right of property in the pavements thus dealt with, and that there was neither justice nor equity in requiring the people who had been at the expense of paving the streets to pay an additional sum for gas because the plant, when put in, would have to be at the expense of taking up and replacing the pavements in building

the same. He held that such added value was wholly theoretical, when no benefit was derived therefrom. We find no error in this disposition of the question."

One of the modern decisions along these lines is that of the Court of Appeals of the State of New York (Kings County Lighting Co., People *ex rel. v.* Willcox, 210 N. Y., 479, March 24th, 1914). The New York Public Service Commission for the First District declined to include in a valuation of the property of the Kings County Lighting Company the pavement over its mains in cases where such pavements were not laid at the expense of the company.

The Appellate Division of the Supreme Court of New York reversed the determination of the Commission. The case was taken to the Court of Appeals, which reversed the action of the Appellate Division and sustained the ruling of the Commission. Judge Miller says (at pages 494-495):

"In determining the cost of reproduction, the Commission allowed $12 717 as the cost of restoring the pavement as it existed when the mains and service pipes were laid in the streets. The relator claimed an allowance of at least $200 000 for the cost of restoring pavements subsequently laid, on the theory that the cost would have to be incurred if the mains were to be laid to-day. But the new pavements in fact added nothing to the property of the relator. Its mains were as serviceable and intrinsically as valuable before as after the new pavements were laid. The controlling considerations under the preceding point also determine this. The rights of the public are not to be ignored. The question has a double aspect. What will be fair to the public as well as to the relator? (Smyth *v.* Ames, *supra.*) Should the public pay more for gas simply because improved pavements have been laid at public expense? It is no answer to say that the new expensive pavements suggest improved conditions which, though adding to the value of the plant, will not, by reason of the greater consumption, add to the expense per thousand feet of the gas consumed. The public are entitled to the benefit of the improved conditions, if thereby the relator is enabled to supply gas at a less rate. The relator is entitled to a fair return on its investment, not on improvements made at public expense. It is said that the mains will have to be relaid. So will the new pavements, and much oftener. Both might possibly be relaid at the same time. The case is not at all parallel to the so-called unearned increment of land. That the company owns. It does not own the pavements, and the laying of them does not add to its investment or increase the cost to it of producing gas. The cost of reproduction less accrued depreciation rule seems to be the one generally employed in rate cases. But it is merely a rule of convenience and must be applied with reason. On the one hand, it should not be so applied as to deprive the corporation of a fair return at all times on the reasonable, proper and necessary investment made by it to serve the public, and on the other hand it should not be so applied as to give the corporation a return on improvements made at public expense which in no way increase the cost to it of performing that service.

"The Appellate Division felt bound by the decision of the United States Circuit Court in the Consolidated Gas Case (157 Fed. Rep., 849), and it is true that such an allowance was made in that case. But the United States Supreme Court held in that case (212 U. S., 19) that the rate established was not confiscatory, and did not pass on the propriety of that allowance. What was said in the opinion on the subject of present value was merely a general statement having no necessary relation to the question now under consideration."

The more recent decisions of the California, Illinois, Indiana, Maryland, New Jersey, New York, Washington, and Wisconsin Commissions have excluded from "fair value" the cost of pavements laid without expense to the company.

This subject is discussed further under the caption "Shall Present or Original Physical Conditions Govern?" on pages 1363 to 1370.

Under special laws providing for the creation of some public utilities, of which elevated railroads and large storage reservoirs are examples, it is required that the owner of the utility shall pay incidental damages on account of property injured, but of which no part is taken. Such damages are an important factor in the original cost of a property, and a corresponding item should appear in the reproduction cost, irrespective of the fact that the owner of the public utility has no title to the property in question.

Working Capital.

It is customary to include under the term "working capital" the amount of cash, materials and supplies provided for use in the plant but not yet forming a part of it, and other current assets which are essential for the proper maintenance, operation, and administration of a property.

There should be included in a valuation an amount of working capital sufficiently large, not only to meet the usual requirements, but to provide for emergencies.

Securities Owned.

Ordinarily, the valuation of property devoted to public use should not include securities owned or surplus cash not forming a part of working capital, except in instances where such securities and surplus cash are an offset, in whole or in part, for depreciation deducted from the cost of the property. The treatment of such cases is covered in Chapter VI, "Depreciation and Appreciation."

CHAPTER IV.

ORIGINAL COST TO DATE.

The term "original cost to date" is used in the Federal Valuation Act, and appears to have substantially the same meaning as the terms "original cost", "original cost plus improvements", and "actual cost".

In the absence of any generally accepted or well-defined legal meaning of this term, the Committee has defined it as "the cost of the original construction, plus all charges against capital proper, under approved accounting principles, for expenditures incurred thereafter, and minus all proper credits to capital for the cost of property which has been disposed of or otherwise retired".

Under this definition, the original cost to date of a property is the first cost* of the identical property units now in use, including overhead charges. This definition accords with modern methods of accounting, by which the cost of property retired is credited to the fixed capital account† to which it stands charged, or to some corresponding account, and the cost of property, added as a replacement, or otherwise, is debited to the fixed capital account or other corresponding account; it seems, also, to conform more nearly than any other definition with the decisions of the Courts that property which has been retired shall be excluded (excepting its salvage value) when making a valuation.

In the case of properties built by present owners, original cost to date should agree with the book or accounting cost in most respects, if modern methods of accounting have been followed from the beginning, but, even with such properties, correct accounting in accordance with modern practice is rare, and book cost, even when the books have been well kept, may differ greatly from original cost to date, as above defined.

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* The definition of cost given by the Committee includes "not only money outlay, but also the money value of services rendered and of other considerations involved".

† In "Uniform System of Accounts for Telephone Companies", as prescribed by the Interstate Commerce Commission, First Issue, Effective January 1, 1913, fixed capital is defined as "property, both tangible and intangible, which is devoted to the accomplishment of the principal purposes of its business", and it "consists of original capital, additions, betterments, and replacements".

Referring to replacements, which it defines as "the substitution of one building, structure, piece of equipment, or machinery for another which it has become necessary to retire, the substitute having substantially no greater capacity than the properly replaced", it provides the following method of accounting:

"The cost of the fixed capital retired should be credited to the fixed capital accounts in which it is carried and the cost of the fixed capital installed in place of fixed capital so retired should be charged to the appropriate subaccounts under account No. 101, 'Fixed Capital Installed Since December 31, 1912'."

Under the head "Fixed capital withdrawn or retired", it provides:

"When any tangible fixed capital acquired subsequent to December 31, 1912, is withdrawn or retired from service for any cause the amount at which it stands charged should be credited to the fixed capital account in which it is charged, and such amount, plus the expenses incident to the retirement, less the value of salvage, should be charged to account No. 102, 'Reserve for Accrued Depreciation—Cr.'"

One important difference between book or accounting cost and original cost to date has arisen from a former method of bookkeeping which is still in use for some classes of property, by which, when a plant unit is replaced by an identical unit, no entry is made on the books, and consequently the cost of the original unit is retained and not the cost of the succeeding unit.

For instance, if a pipe costing $20 per ton for material is replaced by one of the same size and weight costing $25 per ton for material, the cost at $20 per ton remains on the books. Similarly, if a plant unit is replaced by one of larger size or heavier weight, the cost of the additional size or weight is entered on the books at the prices existing at the time of the replacement, and added to the cost of the original unit, so that the new unit appears in part at one price and in part at another.

Difficulty in Obtaining Original Cost to Date.

The statement has frequently been made, by those dealing with the valuation of public service properties, that it is not feasible to obtain the original cost to date of most old properties, and this is undoubtedly true in most cases in regard to the older portions of such properties and to the overhead charges. The difficulty, however, has been magnified in some cases by the conception that original cost to date deals with the money paid for original property units which have been replaced, and not with that paid for existing units. In some cases much of the difficulty may be removed under the definition given by the Committee, especially in the case of short-lived property.

As original cost to date, with comparatively few exceptions, is not the book cost of the property but the cost of the existing items, it will be necessary as a rule to make a schedule of the various existing property items, in the same way that one would be made for determining the cost of reproduction; then reference would have to be made to the accounts, to ascertain the unit costs of the items.

It would seem that almost any system of accounts which was correctly kept, even if not based on modern accounting principles, should show the cost of those items which have been purchased in the last 10 to 20 years. The cost of longer-lived items may not appear on the books, or the books for the earlier years may not be available, especially if the properties have changed hands. In such cases, the only way in which the determination of original cost to date could be completed would be by estimating, from the best information which could be obtained, the unit prices existing at the time the property items were constructed or acquired.

In the case of an old property consisting mainly of long-lived items, it is seldom practicable to obtain the original cost to date, except for the additions made in recent years, and if such cost could be obtained it would have little value.

In every case where records of original costs of presently existing items are not to be had, an attempt to determine original cost to date becomes an estimate, the correctness of which will depend on the completeness and accuracy of existing historic data. The result is likely to be less inaccurate in the case of a recently built property than in the case of an old property, but in no case in which estimate must be resorted to can the result properly be called original cost to date.

Schedule.

To ascertain original cost to date, it is necessary, as a rule, that a schedule be made of the various existing property items in the same way that one would be made for determining the cost of reproduction. (See further discussion of "Schedule" on pages 1377 to 1380.)

Exceptional cases, in which a detailed schedule may not be necessary for all parts of a property, include those in which the whole or a part of a property has been built at so late a date that the original units are still in existence, and those other properties of comparatively recent date where the accounting has been such that additions and betterments are charged to the fixed capital account and the cost of property retired has been credited to this or a similar account. However, even in these cases, an inventory may be desirable for the purpose of checking the results obtained from the books. Under the most favorable conditions as to records, it may be necessary to make many adjustments in order to obtain correct results.

Costs—Unit Costs.

In cases where a schedule is necessary, the corresponding costs or unit costs are also necessary in order to complete the inventory. Whether these are the actual costs of units or groups of units, or the unit costs, will depend on the method of bookkeeping which has been used.

There may be many cases in which work which covers many items has been done by a contractor for a lump sum, in which case it may be necessary, in making the schedule, to group the items included in the contract and to place opposite them the total sum paid the contractor, together with all the incidental charges properly attributable to the work.

When practicable, however, it is preferable to keep the items on the schedule separate, or in groups of identical items of the same age,

and to apply unit costs, either given directly in the books or which may be determined from them, including in such costs the direct incidental expenses, such as inspection, freight, carting, storing, laying or placing, etc.

In the case of property units acquired or created long ago, it will be impracticable, as a rule, to obtain the costs or unit costs from the books, and in such cases the ascertainment of the original cost is impossible, and it would have little significance if it could be ascertained.

Overhead Charges.

The expenses to be included in overhead charges are discussed more fully under this head in the succeeding chapter relating to cost of reproduction.

In brief, however, they may be said to consist of the preliminary expenses of promoting, investigating, organizing, and financing, and the incidental expenses during construction, which may be classified under the heads of engineering, general expenses (including the administrative, legal and financial expenses, taxes and insurance), and interest.

When the detailed accounts giving the cost of a property are available, these items should appear in them, but in many cases no account may have been taken of the value of the services of those engaged in promoting and constructing the work, or proper allowance for interest on the capital, or the records may be in such shape as to defy analysis.

Bonds may have been sold at discounts. Under the ruling of the Interstate Commerce Commission, that proportion of discounts and commissions on bonds is to be included in capital which is represented by the proportion of the construction period to the life of the bonds. The remainders of the discounts and commissions are not to be considered as capital, but are to be amortized over the remaining life of the bonds.

The Committee believes that reasonable commissions for selling securities of any kind should be included as a capital charge, but that discounts on bonds over and above such reasonable commissions should be treated as a capital charge only in such proportion as the construction period bears to the life of the bonds, the remainder being amortized over the remaining life of the bonds.

After the construction period has ended and the regular operation of a property has begun, the method of accounting is frequently such that the salaries and expenses of the permanent engineering, administrative, and legal staff are charged wholly to operating expenses, although a part of the time of this staff is devoted to the construction of additions to the property, and should properly be included as a part of

their cost. Similarly, there is frequently a failure to charge to the proper account the interest during the construction of such additions. Adjustments should be made so as to include in the cost of such additions the overhead charges properly attributable to them. On the other hand, there may be occasion, when determining the overhead charges from the book accounts of an old property, to make deductions of such charges as are properly attributable to plant units which have gone out of use.

The statements in the last paragraph may invite the criticism that, if the method of bookkeeping is such that a part of the overhead charges for additions to the property has been placed in operating expenses, it has been paid for by the public and not by the company, and hence should not appear in a valuation of the property. In this matter the Committee makes a distinction between valuation and regulation, and takes the ground that in valuation the total cost of the property should be included, regardless of past methods of bookkeeping. It recognizes, however, that in continuous regulation, including the control of rates by a public service commission, there should be no duplications by charging items to both the capital and the expense accounts.

Development Expense.

The development expense actually incurred in connection with the tuning up and creation of the business of a property should be included as a part of the original cost to date. The subject is fully discussed in Chapter VII.

CHAPTER V.

COST OF REPRODUCTION.

The Theory of Reproduction.

Different Applications of Reproduction in Past Practice.—Cost of Reproduction is defined by the Committee as:

"The estimated cost of reproducing the property without deduction for the loss of value due to age or other causes."

The practice of those engaged in valuation work, from the beginning of such work up to the present time, has varied widely in the matter of determining the cost of reproduction. Some base such cost on existing physical conditions, others on historic conditions, and still others combine the two. Some engineers have included only those physical property units which were actually created in the construction of the property, that is, they have used historic conditions, as to items of cost, with present-day prices for labor and material. Others have used substitute units, or historic prices, or original instead of present methods of work, and still others have used original conditions, original prices, and original methods, in making an estimate of reproduction cost.

This failure of engineers engaged in valuation practice to agree on a uniform conception of reproduction has cast some doubt on the real worth of Cost of Reproduction as one of the measures of value.

The United States Supreme Court, in the Nebraska rate case, states the various matters to be considered in determining value, as follows: (Smyth *v.* Ames, 169 U. S., 466, March 7th, 1898).

"And in order to ascertain that value, the original cost of construction, the amount expended in permanent improvements, the amount and value of its stocks and bonds, the present as compared with the original cost of construction, the probable earning capacity * * * are all matters for consideration and are to be given such weight as may be just and right in each case."

This case has been cited in most valuation cases, and the use of the word "Reproduction" has been very general in subsequent decisions, although it is to be noted it is not used in the decision in Smyth *vs.* Ames. It is clear that the Courts in many cases have recognized the reproduction estimate as one of the most important bases for determining value (Louisville & Nashville R. R. Co. *v.* Railroad Commission of Alabama, 196 Fed., 800 (1912), Hon. Thomas G. Jones, District Judge, N. & M. D., Alabama).

"In reference to the question of value with the view of rate regulation, the most reliable test ordinarily is the cost of reproduction of

the road as it exists. I say 'ordinarily' because there may be instances, which is not the case here, where by reason of paralleling the road by a new road, and diverting its business or from other causes, its value may be far less than what it will cost to reproduce it as it is at the time of the inquiry. The original cost of the road may in some cases reflect light on, or even determine, the present value, as when it is of very recent construction. But ordinarily it is of little assistance in that regard, since many items of value may be donations by the government or by individuals * * * or the road may have been built long before the period of inquiry at greatly less or greatly higher prices than those prevailing at the time of the inquiry. * * * Therefore, while looking at all collateral matters reflecting light on the subject, the court regards reproduction cost as the final test of present value. * * *"

In the Knoxville decision it is stated that:

"The cost of reproduction is one way of ascertaining the present value of a plant."

In this and practically all other cases in recent years, in which the values of old properties were under consideration, the evidence relating to cost which has served as a basis for determining value has been the reproduction cost.

In the case of the Consolidated Gas Co. *vs.* City of New York (157 Fed. Rep., 849, December 20th, 1907), District Judge Hough says:

"The complainant demands a fair return upon the reproduction value thereof, which is the same thing as the present value properly considered. * * * Upon authority, I consider this method of valuation correct. What the court should ascertain is the 'fair value of the property being used'; what complainant 'employs for the public convenience'; the 'present' as compared with 'original cost' (Smyth *vs.* Ames, 169 U. S., at page 547 * * *); and it is also the 'value of the property at the time it is being used' (San Diego Land Co. *vs.* National City, 174 U. S., at page 757 * * *, and see also Stanislaus Co. *vs.* San Joaquin Co., 192 U. S. 201 * * *). It is impossible to observe this continued use of the present tense in these decisions of the highest court without feeling that the actual or reproductive value at the time of inquiry is the first and most important figure to be ascertained."

In this decision value is used in two senses: one, value as the Committee has defined it; and two, as synonymous with cost.

In the Minnesota rate cases, it is stated that:

"The cost of reproduction method is of service in ascertaining the present value of the plant when it is reasonably applied, and when the cost of reproducing the property may be ascertained with a proper degree of certainty. But it does not justify the acceptance of results which depend upon mere conjecture."

This statement follows a discussion of the cost-of-reproduction method as applied to the determination of the value of a right of

way, which consisted largely of terminal property, in which it was held that the cost of reproduction did not give proper results.

This case sounds a note of warning against the use of irrational, or unsound, theories of reproduction, and the use of mere conjecture in making a reproduction estimate.

The Supreme Court has held consistently, in the older cases, that in determining value, original cost under original conditions, and reproduction cost under present conditions, were of the greatest importance, though other considerations, such as selling (market) price of the securities, worth of the service to the consumer, etc., were also of significance. In these older cases the Courts seem to have said that reproduction cost should be determined on the basis of present-day conditions, whether or not the corporation years ago had to take a series of steps involving expense which would not now have to be taken, or whether or not the property was built by piecemeal construction at a necessarily higher cost than would prevail if all built at one time; that "present-day conditions" means present-day prices, present-day methods, and present-day facilities for doing work, and that these should be applied under existing physical conditions.

With the original cost and reproduction figures in hand, determined as outlined above, and with such other information, historic and otherwise, as was available, the Courts have held that valuation should be summed up as a matter of judgment, giving due weight, not only to the results obtained by the application of these different measures, but incidentally to the degree to which the measures are equitably applicable.

The more recent and authoritative decisions of the higher Courts, though they still lean toward defining the cost of reproduction as the cost under present conditions, thus following precedent, also give strong support to the use of historic physical conditions, since, in determining the "fair value" of pipes originally laid in streets having no pavements, but in which pavements were subsequently laid at the expense of the city, they exclude from "fair value" the cost of taking up and replacing the pavements which would be included if one were to base "fair value" on the cost of reproduction under existing conditions. In other words, the sums allowed by the Courts in determining "fair value" are the sums which it would cost to lay the pipes under the physical conditions which existed at the time the pipes were laid. The subject seems to be in a developmental stage in the Courts.

In effect, the Courts in some instances appear to have recognized that a split standard, so to speak, or one which might be defined as reproduction cost under present-day material and labor prices, and historic or past physical conditions, in certain particulars, would do more substantial justice, or be more equitable in the results of its

application, than either the original-cost theory under historic or past conditions on the one hand, or the reproduction-cost theory under present conditions on the other. Under such a standard the owner enjoys the advance in general value standards indicated by the advance in cost of labor and materials, or suffers from the decrease thereof, if these have declined in value, and has recognition given to the conditions under which he made his investment.

May not the question fairly be raised: Has the conception of reproduction, based on present conditions, been a proper conception?

In the opinion of the Committee, this subject is of sufficient importance to justify full discussion of the question: What is the proper conception of cost of reproduction new? In this discussion the Committee will point out some of the difficulties to be met in formulating a rational conception of the problem, will then try to answer four questions, and finally draw its conclusion.

Difficulties to be Met in Formulating a Theory.—The original conception of reproduction undoubtedly grew out of simple conditions, such as would be met in estimating the cost of reproducing a building or a single structure. In this case, the difference between the original cost of the building or structure and the reproduction estimate would be due wholly to changes in prices of labor and materials and the change in methods of doing work. This simplicity of condition does not obtain in the case of a great property, the actual construction of which has extended over many years, many of the plant units of which have been renewed or replaced by larger ones than were originally installed, which has undergone changes and alterations, and the history and records of which have not been kept fully and completely. In such a case the making of a complete estimate of the cost of replacement or reproduction is a very involved undertaking.

When there are added the complexities due to the effect of works built by other companies than the one under investigation, or by municipalities, or to the growth of a community or rearrangement of transportation facilities of a community or section of country, elements are introduced, the proper treatment of which is vital to a reasonable estimate of the cost.

To obtain the reproduction cost of the property, shall it be assumed that all those things must be done that are or were necessary to create a new property in the same location as that of the existing one? It is usually assumed that all other existing properties or conditions affording facilities for, or offering difficulties to, the reconstruction of the property under consideration continue to exist as at present, in so far as they affect the cost of reconstruction; but one cannot forget that the property in question was not built in a day—nor during one construction period.

Nearly all the second, third, and fourth tracks, and side tracks, of steam railroads have actually been built years after the original construction, with certain disadvantages arising from maintaining traffic but with the advantages to be derived from the existence of a well-organized railroad over the entire route. The same thing is true of the construction of many existing structures, such as large buildings, bridges, signal and interlocking systems, and other more recent and modern adjuncts of the railroad systems of to-day.

The actual historic development of railroads has seldom been the springing to life of a new complete up-to-date property, but a growth over a long period of years, each company adding to its track and facilities as the needs of its territory and its business demanded. The same thing is true of water-works, gas, or electric properties, and other utilities in large cities. Under these conditions, it is clear that great care must be exercised in outlining the basis for estimates.

The four main questions to be asked and discussed are:

1.—Shall present or original physical conditions govern?
2.—Shall present or original prices govern?
3.—Shall identical or substitute plant be considered?
4.—What is meant by the word "new", as frequently used in connection with reproduction?

1.—Shall Present or Original Physical Conditions Govern?—A few examples of actual conditions encountered in connection with construction are presented.

Wachusett Reservoir.—This reservoir was built to supply water to the Metropolitan District of Massachusetts. To construct it, it was necessary to acquire land on which there were many buildings, and mills with their water powers. Included in the reservoir site there were also many highways and two railroads. As a substitute for the highways obliterated, others had to be built around the margin of the reservoir, and some were raised above the water level. One of the railroads was relocated for many miles at one side of the reservoir, and another was raised. To make the reservoir a better receptacle for water, the surface soil was stripped from its whole area. One main dam and two subsidiary dams were necessary for holding the water. Incidentally, under special laws, damages had to be paid for real estate which was not acquired, but which was said to be damaged indirectly by the construction of the reservoir.

Kensico Dam.—This dam, of the New York Water-Works, was necessarily located within the limits of an existing reservoir, the water of which could not be drawn down until two new temporary reservoirs had been created farther up stream to maintain the water supply. After these had been completed, the old Kensico Reservoir was drained and the old dam removed. After the completion of the

new Kensico Reservoir, the two temporary dams and reservoirs will be useless.

The AuSable Dams.—By building three dams on the AuSable River in Michigan, the Commonwealth Power Company flooded hundreds of acres to a depth of 40 ft. There was not a house, not a mile of road, no railroads, no damage to property of any kind such as was encountered in the other reservoirs. It was only necessary to acquire the needed lands and flowage and build the dams, without property damages in any appreciable amount.

There are now no buildings, roads, or railroads in the basins of any of these reservoirs, nor does the connection of the Kensico auxiliary reservoirs with the construction of the main dam appear. Therefore, in these three cases, presently existing conditions are not indicative of work done or difficulties encountered in actual construction in the past, nor of what it is fair to assume would be the conditions to be found had the reservoirs not been built. Conditions may be now essentially similar, immediately about these reservoir sites, but it would seem to be unfair for this reason to assume like conditions to govern the estimate of reproduction cost to be used as a basis of "fair value".

The theory of reproduction cost under existing conditions evidently was based on the reproduction of property units which now exist, such as, for instance, buildings, bridges, and other structures and works, and did not take into account that, in creating public service property, there may be much destruction as well as construction. The destroyed property obviously does not exist, and therefore would not be included in the inventory of property to be valued if the theory of determining the cost of property under existing conditions were followed strictly.

In addition to the destruction of physical property, cited in the foregoing, there may be the destruction of business and resulting damage to surrounding property, all of which may prove to be a large element of cost.

On account of the destruction of business, it became necessary in the case of the Wachusett Reservoir to pay for the resulting loss, not only to those owning the business, but to the workmen who were thrown out of employment, and to the owners of surrounding property not acquired for the reservoir but affected in value by the destruction of the manufacturing and other business in a well-populated valley. It is clear that such business will not exist at the time a reproduction cost is being ascertained, and therefore will not be an element of cost under existing conditions rigidly interpreted. The destruction of the business was properly an important element of cost in the creation of a reservoir at this site, would be reasonably certain to be an element of cost in the production of a reservoir at this site at any date of

estimate, if the existing reservoir had not been built, and therefore should be included as an element of cost of reproducing the reservoir.

Your Committee is of the opinion that all such items of cost, due to damages, destruction of property, purchase of rights, and temporary works, as cited in these illustrations, are clearly proper items to be included in the reproduction estimate when capable of historic proof. It would seem that, in such cases as these, fairness to the owners of the property and to the public would necessitate the use of historic conditions, at least to determine the extent, magnitude, and character of property units created, destroyed, or removed, physical evidence of which passed with the construction of the property itself, and which it may reasonably be assumed it would still be necessary to create, destroy, or remove were the property to be built new at the date of valuation.

Another series of examples may be found in the conditions which existed at the time of construction of certain great railway terminals.

The Pennsylvania Railroad Terminal, New York City.—This structure was pioneer construction. Land was purchased extending in a strip across the greatest city in the country. The property was built up solid with buildings, which were not only purchased, but wrecked, and the entire value thereof thrown away. There was no steam railroad traffic to maintain, but there was consequently no benefit to be derived from the existence of tracks for the removal of earth and construction débris.

The New York Central Terminal, New York City.—This terminal replaced an old structure, which in turn had succeeded others on the same site. The existing plant covers an extensive area, the larger portion of which was originally, with respect to the present undertaking, covered with buildings of all kinds and intersected with streets requiring treatment in various ways, as in the case of the Pennsylvania terminal, but in building this terminal, the company maintained a formerly created traffic without interruption, and though thereby the expense of construction was increased by the cost of temporary structures to provide facilities for its patrons, longer time of construction, and cost due to the hampering of operations by not having the entire site available for working purposes, it is probable that these elements of additional cost were largely offset by the savings that resulted from the utilization of the existing accessorial tracks for the handling of construction materials and the disposition of excavated materials in four-tracking the Hudson and Harlem Divisions.

Except with respect to the value of near-by lands, the present conditions about these two properties are much alike, and, for anything that appears, a reconstruction programme for one would be much like that for the other, under present surrounding conditions, and the offsetting effect of the revenue earned by the New York Central Railroad, against its increased cost due to maintenance of traffic, gives

color to the propriety of making like assumptions for both properties. But it should be remembered that only a part of the property of the New York Central Terminal had to be acquired in modern times. Much of it was acquired years ago, when it was not covered by valuable buildings, and it would seem to be unfair to assume, in a reproduction estimate which is to serve as a basis for "fair value", that all of it would be covered with valuable improvements, to be purchased and wrecked simply because surrounding land is now so covered, or because the property acquired by the Pennsylvania Company was so covered. Without the existence of the railroad, the City would not be the city that it is; it is what it is, partly as the result of the existence of the railroad, therefore it cannot be said that if the railroad had not been built, the surrounding lands, and presumably the site of the terminal, would be covered with the valuable improvements now in evidence. Here again it would seem to be fair to refer to history to determine what was done, and would reasonably be supposed still to be necessary to do, to construct the property new.

One illustration brought to the attention of the Committee is a case of a highway which originally crossed a steam railroad at a very acute angle, making a dangerous crossing to both railroad and public. At its own expense, the railroad acquired land parallel with its own, changed the course of the road for several hundred feet and made a right-angled crossing. It was argued that, "in case of reproduction, all other property remaining as at present", the highway would be in its new location and would not have to be reproduced. On this basis, many items which the owner of a railroad or other public utility was compelled to pay for might be cut out of an appraisal. But the highway would be in its original place were it not for the act of the railroad, and would have to be moved by any railroad, building presently, if the existing road had not been built; the evidence of the necessity, however, having been destroyed in the doing. The cost was a proper cost, capable of historic proof; and the Committee believes that it should be included as a cost in the estimate of reproduction.

A certain railroad was built through a heavily forested section of country, largely to develop the lumber industry in that section. The land was worth, and cost the railroad company, upwards of $40 an acre because of its timber value, and the company paid another $40 or $50 an acre, for clearing and grubbing. The timber industry developed, the surrounding lands were cleared and proved to be unattractive for agricultural development, and no large cities built up along the railroad. To-day the timber is gone, there is no clearing to do, and the land has depreciated to a value of $5 or $6 an acre. In this case human progress has taken away from the value of the territory, rather than, as is more usual, added to it. The railroad has

suffered as well, because its local business is largely gone. It has, however, as a part of a large property that has taken it over, developed a good through business. It would seem that all these things must be considered, when determining whether the original conditions for right of way and clearing, capable of proof, shall be used in an estimate of reproduction cost, or on the other hand what would be the present-day cost of acquiring the already cleared right of way. It is manifest that the railroad company contributed to the present condition, by providing the means for removing the lumber. Presumably its promoters should have been able to foresee what the general result of the lumbering operations would be, and should have reimbursed themselves, in the early days of the enterprise, for the possible future loss in value, due to the changed conditions, and in general it must be said that such enterprises assume the hazards of such results of human progress, particularly those to which they themselves contributed largely. Nevertheless this line, while apparently not now needed for the country through which it operates, and which, but for its connection with a greater property, would certainly have to be depreciated in value by at least the diminished cost of reproduction; yet, as a going property, adequately serving the public in its through business, it would seem to be unjust to take away from it all hope of future life as might result from a limitation of its return to "fair return" on a greatly depreciated actual investment.

One element of cost, being typical of a class and easily separable, deserves discussion by itself. This is the cost of removing and replacing paving of streets over and around public utility properties. It is of importance in connection with street railways and all underground conduit lines for gas, electricity, water, or other commodities.

Street railway properties occupying the streets of large cities do so under grants which vary widely in terms as to paving. In every case the burden is on the company to build such a type of construction as will permit that part of the public not depending on the railway company for transportation to enjoy the free and unrestricted use of the street.

A certain street railway company owns a large mileage of track in paved streets. In addition to the not uncommon case of tracks which were laid before the streets were paved, several different cases of complication as to paving are found:

Case 1.—The street railway company built all the track foundations, track superstructures, and pavement.

Case 2.—The street railway company built all the track foundations and superstructures, but the city built the pavement.

Case 3.—The street railway company built the track superstructure, but the city built the track foundations and pavement.

Case 4.—The street railway company built the track foundations and superstructure and took out and rebuilt existing pavements, but subsequently the city repaved the entire street at no expense to the company.

In making an appraisal of this property it will be necessary to define clearly what items shall be included in the reproduction estimate, and to answer, among others, these questions: Shall allowance be made for the removal of paving on all streets, whether they were unpaved or paved when the tracks were built? Shall allowance be made for all track foundations, or shall investigation be made as to what was done by the city, and that part excluded? Shall allowance be made for paving in all streets in which there is paving?

The Committee is of the opinion that in all cases where such questions are at issue, the terms of the contract should govern. The property built by the corporation under the terms of its charter or franchise, whether the items belong to the corporation or the public, is the property to be reproduced. Franchise or contract provisions usually settle all questions of ownership or of division of joint expense. Such provisions should govern as to the inclusion of all, or part, of the expense in the reproduction estimate. It is immaterial whether another corporation or the public is the other joint owner. The division should be made on a proper interpretation of contract or franchise, and in such cases as those cited, no expense to be borne by the city under the contract should be appraised to the railway company, and no expense to be borne by the railway company under the contract should be omitted in determining the reproduction cost. The position taken in some cases, that such an expense as paving is not a permissible capital charge and therefore not an item of cost of reproduction in the case of a street railway, does not seem to be tenable when the expense is borne by the railway company.

When a corporation is required, as is usual, to remove and replace paving when constructing works requiring such removal and replacement, there can be no doubt that the existence of pavement in a street makes the construction of tracks, or the laying of pipes or conduits, more costly than if the pavement were not there. When pavement exists, therefore, at the date of estimate, its effect on the cost of reproduction cannot be ignored, even though the extra cost should not be included when using the reproduction cost as an element of "fair value". The item when included should be scheduled separately, so that it can be omitted if that be the will of the determining body.

This subject has already been referred to under the caption "Title to Property Not Conclusive", on pages 1350 to 1353.

The item of clearing and grubbing railroad rights of way is one concerning which question has been raised. On few if any rail-

roads, is there any forest on the right of way. Along many miles of existing railroad there is to-day no evidence that there ever was any forest growth on the right of way, where history shows the existence of dense forests at the time of original construction. It is true that this is a minor question, because the rise in land values, if allowed, will in general—but not always—more than offset any possible loss to the company-owner by reason of the exclusion of the item of clearing and grubbing actually paid for, but no longer in evidence as ever having been necessary. The fact, that there may be offsetting advantages and disadvantages, does not affect the principle involved. The effort is to get a basis for the present-day value of the owners' investment. If, as a result of human progress, it would be reasonable to suppose that, at the date of valuation, the land occupied by the railroad would have been cleared, regardless of the existence of the railroad, and it is so cleared, it would seem to be fair to omit clearing from a reproduction estimate. If, on the other hand, it is fair to assume that, but for the presence of the railroad, the forest would still be there, the cost of clearing and grubbing should be allowed. There are cases in which one, or the other, of the assumptions herein made is clearly the correct assumption, there are other cases in which it is not so clear, as to whether the changed conditions are due to the railroad itself. Let it, as the party bearing hazards, be given the benefit of the doubt.

Another class of expenditures, that may be greatly affected by the choice of historic or present-day conditions, is that including promotion, organization, financing, interest during construction, and some other overhead charges, development expense, and the excess cost of piecemeal construction.

There will be no difficulty with respect to newly created properties. The question occurs with respect to old properties of magnitude, that have grown from relatively small beginnings.

In such cases most of these expenses would be relatively large for the original enterprise, and relatively small, or nothing at all, for the smaller additions as they have been made from time to time—although interest would always be an expense, and the others named would be incurred in greater or less degree on all but minor additions. And it may very well happen that important additions may be of such relative magnitude that all of this class of expenditures will be as large, in percentages of the property to which they apply, as were the original expenses of this class of the original property.

It would seem, therefore, that with respect to these items, the policy should prevail, of referring to the history of the property, to determine to what portion of the property, and how these items of expense, are properly applicable, just as has been advised for the determination of those items of physical property that are to be in-

cluded, not forgetting that at least the item of interest is applicable to the whole of the construction, but for varying lengths of time, and that development expense attaches to practically all additions of any considerable relative magnitude, and the excess cost of piecemeal construction to all minor additions.

Concerning facilities for doing work in general it would seem to be logical to use those of the period of valuation, but with respect to transportation agencies, this may work injustice in some cases. The Los Angeles Aqueduct, and certain other works, all built without near-by railroads to bring materials, men, and supplies to the site of the work, could now have the advantage of more recently constructed railroads. Have these great works, fully serving their several purposes, lost a considerable portion of their value by the development of transportation facilities not in existence when they were built? It would seem fair to say "no", even though this may seem to be inconsistent with the position taken with respect to most old properties. The uncertainties with respect to the details of facilities existing, or used in the construction of old properties, and the effect of these on unit prices, which, as will appear later, the Committee thinks should be as of the period of valuation for reproduction estimates, make it seem desirable and consistent to use present-day transportation facilities for most old properties, and historic facilities for some exceptional cases of old or recently constructed properties.

The conclusion of the Committee is that, while apparent present-day conditions, that would affect the cost of reproducing the property, must be considered in any logical estimate, yet history must also be considered to determine what is to be reproduced, the conditions under which it is to be reproduced, and how the estimates must be made; that for all those items, concerning which there can be no doubt, the engineer should use the basis plainly applying, and that for those that are doubtful, or have been questioned, he should present the effect of the use of the different bases clearly, that the determining body may have the data for a wise decision.

2.—Shall Present or Original Prices Govern?—Prices for labor and materials rise and fall, and the cost of doing work varies, as new machinery and appliances are invented.

The term "present prices" has sometimes been held to mean the prices of the day of estimate; sometimes the average prices of a greater or less period just preceding the day of estimate; and sometimes the prices of a similar period just following the day of estimate. By the term "present prices" the Committee means prices averaged over a period of time to produce what may be called normal prices, as is fully discussed under the caption, "Unit Prices".

One of the recognized bases of "value" of property is the sum that it would cost to produce that property or an exactly identical

property at the present time. Estimates of the "cost of reproduction" are made to show what the property could be produced for new at the time of appraisal, and therefore unit prices which were actually paid many years ago may not now apply. It would seem to be clear that, in estimating the cost of producing any property at a given time, unit prices which do not and cannot prevail at the time of the investigation should not be used; and, conversely, it would seem to be equally clear, without argument, that the prices for materials and labor should be those of the assumed time or period of construction.

Important decisions of the United States Supreme Court, on which the Committee bases its conclusion, that present prices for materials and labor should be used, are Willcox *vs.* Consolidated Gas Company and the Minnesota rate case. In the former, Mr. Justice Peckham says (212 U. S., 19, January 4th, 1909):

"And we concur with the court below in holding that the value of the property is to be determined as of the time when the inquiry is made regarding the rates. If the property which legally enters into the consideration of the question of rates has increased in value since it was acquired, the company is entitled to the benefit of such increase. This is at any rate the general rule."

The decision referred to is that of Judge Hough in Consolidated Gas Co. *vs.* City of New York (157 Fed., 849, December 20th, 1907).

"Upon authority, I consider this method of valuation correct. What the court should ascertain is the 'fair value of the property being used' (Smyth *v.* Ames, 169 U. S., at page 546, 18 Sup. Ct., at page 434 (42 L. ed. 819)); the 'present' as compared with 'original' cost; what complainant 'employs for the public convenience' (169 U. S., at page 547, 18 Sup. Ct., at page 434 (42 L. ed. 819)); and it is also the 'value of the property at the time it is being used' (San Diego Land Co. *v.* National City, 174 U. S., at page 757, 10 Sup. Ct., at page 811 (43 L. ed, 115)). And see, also, Stanislaus Co. *v.* San Joaquin Co., 192 U. S., 201, 24 Sup. Ct., 241, 48 L. ed., 406. It is impossible to observe this continued use of the present tense in these decisions of the higher court without feeling that the actual or reproductive value at the time of inquiry is the first and most important figure to be ascertained, and these views are amplified by San Diego Land Co. *v.* Jasper (C. C.), 110 Fed., at page 714, and Cotting *v.* Kansas City Stock Yards (C. C.), 82 Fed., at page 854, where the subject is more fully discussed. Upon reason, it seems clear that in solving this equation the plus and minus quantities should be equally considered, the appreciation and depreciation treated alike. * * * The so-called 'money value' of real or personal property is but a conveniently short method of expressing present potential usefulness, and 'investment' becomes meaningless if construed to mean what the thing invested in cost generations ago. Property, whether real or personal, is only valuable when useful. Its usefulness commonly depends on the business purposes to which it is or may be applied. Such business is a living thing,

and may flourish or wither, appreciate or depreciate; but, whatever happens, its present usefulness, expressed in financial terms, must be its value."

"As applied to a private merchant or manufacturer, the foregoing would seem elementary; but some difference is alleged to exist where the manufacturer transacts his business only by governmental license—whether called a franchise or by another name. Such license, however, cannot change an economic law, unless a different rule be prescribed by the terms of the license, which is sometimes done. No such unusual condition exists here, and, in the absence thereof, it is not to be inferred that any American government intended, when granting a franchise, not only to regulate the business transacted thereunder, and reasonably to limit the profits thereof, but to prevent the valuation of purely private property in the ordinary economic manner, and the property now under consideration is as much the private property of this complainant as are the belongings of any private citizen. Nor can it be inferred that such government intended to deny the application of economic laws to valuation of increments earned or unearned, while insisting upon the usual results thereof in the case of equally unearned, and possibly unmerited, depreciation."

"I think the method of valuation applied by the report to land, plant, mains, services, and meters lawful."

In the Minnesota rate cases, Mr. Justice Hughes says (230 U. S., 352, June 9th, 1913):

"It is clear that in ascertaining the present value we are not limited to the consideration of the amount of the actual investment. If that has been reckless or improvident, losses may be sustained which the community does not underwrite. As the company may not be protected in its actual investment, if the value of its property be plainly less, so the making of a just return for the use of the property involves the recognition of its fair value if it be more than its cost. The property is held in private ownership and it is that property, and not the original cost of it, of which the owner may not be deprived without due process of law."

The Committee believes that the foregoing decisions require the use of present rather than original prices in estimating reproduction cost. It recognizes, however, that undesirable fluctuations in the estimated value of property valued at intervals would occur owing to changes in prices. It suggests that this may be avoided and the value from year to year of a property which has been once properly valued may be determined if proper methods of accounting are adopted.

In the case of a new or recently created property, which has had from the beginning, under continuous and proper regulation, a modern system of accounting, which has taken account of all proper capital charges and credits, so that the amount of invested capital would be known at all times, such invested capital would be entitled to greater weight on equitable grounds as an indication of the so-called "fair

value" than an estimate of cost of reproduction less depreciation which might involve radical changes of prices; but we are not now discussing original cost to date, nor what is the proper basis for "fair value". For reproduction cost the Committee recommends that, in estimating, the prices prevailing at the assumed time of reproduction shall be used, meaning the normal prices obtained by averaging prices for a proper period, as is fully discussed subsequently in this chapter under the caption, "Unit Prices".

3.—Shall Identical or Substitute Plant be Considered?—Occasionally an estimate has been made, not of what it would cost to reproduce the existing unit, but of what it would cost to produce some substitute unit which would perform the same function as the existing unit.

For example, in a valuation of an old railroad, it was found that all the bridge piers and abutments and retaining walls built in the early years of the property were of stone masonry which would cost to reproduce new from $12 to $16 per cu. yd., and that all work built during the last 10 years was of concrete costing from $7 to $11 per cu. yd. It was argued that the modern construction, built at less expense, fulfilled exactly the same functions, and that, if the property were actually to be reproduced, stone would not be thought of. Undoubtedly, the original construction was used because it was the best available, and, in the judgment of the engineer and directors, this was a proper expenditure.

In one other case an engineer was making an appraisal of a large electric plant furnishing lights to a large city and power to several hundred miles of railroad. The power plant as originally built had been outgrown, and a second plant had been built alongside with about equal capacity. Both plants were working to capacity during the peak load, and both plants were well maintained. It was argued that one modern, up-to-date plant could be built which would cost less than the reproduction cost of the two and perform the same function.

Other illustrations of properties, which have been expanded to meet the need of rapidly growing cities and which in their present form represent very much larger investments of capital than would be necessary were actual new construction to take place at the present time, are afforded by the water supplies of some American cities. One western State capital affords an excellent example. The first water supply was obtained by pumping from a near-by source. Later, a supply was secured some 20 miles distant, but, in order to make use of an infiltration gallery, the water was taken at a point elevated only slightly above the distributing reservoir in the city, so that the capacity of the pipe was small in proportion to its size, on account of the limited fall. Large filter beds were located at this place; still later, an intake was located farther up stream, which made available a greater fall to the reservoir in the city; another feature is a large

natural lake used as a reservoir, fed by a small stream, not very far from the pipe line between the main source and the city, but at a higher level; still later, a great reservoir with a dam more than 200 ft. high was built in the mountains, far above the main source of supply.

It probably cannot be said that the company did not use reasonable judgment in the laying of each of the pipes and the building of the reservoirs and filters referred to, but if one were to design an equally efficient plant at the present day the water could be taken from farther up stream, so as to have an adequate head; it could be brought quite near to the city in a masonry conduit which would cost much less for the same quantity of water than the existing pipes; the lake or reservoir could be utilized at this level as a storage reservoir connected with the main supply; a single filter plant at the proper elevation, much higher than the present one, could be located comparatively near the city as a substitute for several independent filter plants; and two pipes having the same aggregate capacity as all the existing pipes leading from this filter plant to the city would cost much less than the existing pipes because of the much greater available fall. To use the estimated cost of the substitute plant in this city in lieu of the estimated reproduction cost of the present property would ignore a large investment actually and properly made by the water company.

The cost of reproduction being always required for the establishment of some relation between the public and its servant, the service corporation, either with respect to rates for service, purchase price, capitalization, or taxation, it would seem that the company should be credited with what it has actually produced in good faith and with what seemed to be good judgment at the time the property was created.

In the cases cited, the cost of substitute plants or property units altogether different from those actually built would vary very much from the reproduction cost of the existing property, and although such a substitute property, much less costly than the existing plant, might furnish equal or better service, it is not reproduction of service, but of property, that is under consideration; and clearly the estimate should be of existing property created with public approval, rather than of a substituted property.

In the case of the valuation of obsolete units of property still in service, where the unit is of a type not on the market and not possible to reproduce in kind at the present time at reasonable cost, it may be desirable to use the cost of a modern unit capable of performing the same service, but this would appear to be the only exception, and would seem to apply only to individual units, such as machines, which are not capable of being estimated by ordinary standards used in reproduction.

Reproduction in kind deals with facts, with properties that have physical existence and can be measured, inspected, and in many cases

their actual cost determined. Production of a substitute would be based more largely on speculation; different engineers would undoubtedly differ in their views as to the location, design, and cost of the substitute plant, and an estimate based on such a plant would be more difficult to support than one based on an existing plant, by reason of conflicting expert opinion as to the proper procedure.

Although the cost and efficiency of the substitute plant may have some bearing on the value of the existing plant, remote though it may be, it has none on reproduction cost of the existing property.

These views are in accord with the general practice of engineers who have engaged in appraisal work. These engineers have valued the identical property, and this method of valuation, when brought before the various Courts, including the Supreme Court of the United States, has met with approval, although the relative merits of the two methods have not been discussed by the Supreme Court. The principle that the identical property should be valued has been accepted apparently without discussion.

In the case of the Consolidated Gas Co. *vs.* City of New York, Judge Hough says:

"In every instance, however, the value assigned in the report is what it would cost presently to reproduce each item of property in its present condition and capable of giving service neither better nor worse that it now does. * * * Upon authority, I consider this method of valuation correct."

In the case of the Kennebec Water District *vs.* City of Waterville, Judge Savage says (at page 19):

"We think the inquiry along the line of reproduction should, however, be limited to the replacing of the present system by one substantially like it. To enter upon a comparison of the merits of different systems—to compare this one with more modern systems—would be to open a wide door to speculative inquiry, and lead to discussions not germane to the subject. It is this system that is to be appraised, in its present condition and with its present efficiency."

In harmony with the foregoing arguments, it is recommended that, for whatever purpose in connection with the relations of the public and the service corporation the cost of reproduction is required or desired, the estimate should be made with respect to the identical property, and not with respect to some substitute property to perform the same service.

4.—What is Meant by the Word "New", in the Phrase "Cost of Reproduction New"?—The words "Cost of Reproduction New" are of such frequent occurrence in valuation cases, and seem to have given rise to such diverse views, that, before the proper conception can be finally determined, it is necessary to give consideration to the proper weight to attach to the word "New".

For example, a certain logging railroad is 51 miles long. It was laid entirely with relayer rails; ten second-hand locomotives and certain second-hand cars were purchased and have been used for some years. It would be permissible under some conceptions of reproduction to estimate all this rail and equipment as new, but, such a conception would not appear to be proper. The term "new" in a case of this kind would apply to the character and condition of the second-hand materials as they were originally installed, and their depreciation would be measured by the difference between their condition at that time and at the date of valuation. The rail and the equipment are larger and more expensive than the road would be justified in buying if it bought them new. The property as "reproduced", if conceived to be new rail and equipment, is not identical with the original property when it was "new" or first built. There are costs estimated which did not enter into the original investment.

Another example.—On a little draw-bridge a motor has been installed which is in fact an old street railway motor capable of performing its new function indefinitely. It is of much greater capacity than would be needed for the bridge, and if a new motor had been installed a much smaller and less expensive type would have been selected.

In the examples cited it would seem that the properties when created "new" were partly second-hand, and the reproduction estimate should include relayer rails at relayer prices, second-hand equipment and second-hand motor at prices fair for the actual items at second-hand.

The Federal Valuation Act particularly requires that there shall be found: "The original cost to date; the cost of reproduction new; the cost of reproduction less depreciation."

In view of the wording of the Valuation Act, some have assumed that the word "new" requires the assumption of new rail in making a valuation, but that where records show that the material in the road was originally second-hand material, and is at present second-hand material, an initial depreciation equivalent in amount to the difference between the cost per ton of new rail and the cost per ton of relayer rail, such as was used, may be entered under the depreciation column. A further examination of the property itself at date of valuation will determine whether or not there is any additional depreciation, and to what extent it exists.

We speak of a new property and thereby refer to a property which has been created where no such property existed before. It may be of new material in all its parts, or it may be "new" as to grading, ownership of land, and many other of its elements, but have second-hand material and equipment entering largely into its construction.

If substitute units are to be excluded from consideration, consist-

ency demands that original conditions of existing units in their present service must be restored, and that no interpretation of the word "new" is to be permitted which places a property unit in any better or worse condition that it was in when first installed in present service. The property to be reproduced is the existing property. Each unit of property which is in existence is to be reproduced in the same condition as it was when it was put to its present service.

Conclusion as to the Proper Conception of Reproduction.—In line with the foregoing discussion, the Committee recommends that reproduction estimates be based on the assumption that the identical property is to be reproduced, rather than a substitute property; that while apparent present-day conditions, that would affect the cost of reproducing the property, must be considered in any logical estimate, yet history must also be considered, to determine what is to be reproduced, the conditions under which it is to be reproduced, and how the estimates must be made; that for all those items, concerning which there can be no doubt, the engineer should use the basis plainly applying, and that for those that are doubtful, or have been questioned, he should present the effect of the use of the different bases clearly, that the determining body may have the data for a wise decision; and that normal present conditions shall determine the prices and methods for doing the work.

Work Preliminary to the Making of the Estimate of Reproduction.

The Schedule.—The first step in estimating the reproduction cost of a property is such a study of the property and its history as will enable the estimator to make a complete list of all items, lay out a proper financial and construction programme, and fix proper unit prices to the several items of the schedule.

Every appraisal offers its own problems, and it would be impossible for the Committee to lay down any fixed rules for the detail of appraisal that would be of universal or even of general application.

When the records of a company owner—maps, plans, specifications, deeds, and other titles to property—engineering records of construction and accounting records are sufficiently complete, it may be possible and permissible to make up schedules from the records and depend on inspection in the field to furnish a check on the records and to apply information to make good any deficiencies. Where, however, as is often the case, the records are incomplete and fragmentary, it becomes necessary to make most careful field schedules of the property, using the records as far as practicable to check field measurements and to furnish as much light as possible on the history of the construction.

Classification of Property.—For scheduling, the property should be divided into groups of items or units, preferably following some well-

established classification of accounts. For example, in railroad valuation, the grouping of items in each schedule should follow the grouping in the corresponding account of the Interstate Commerce Commission's Classification of Property chargeable to capital accounts. Where units of property or items of cost appear to have no place in such classification of accounts, but are clearly part of the cost of the property or are claimed as such, they should be set up as a sub-group, in the account that appears to be most appropriate.

Field Schedules.—When it becomes necessary to make surveys or field inspections in order to prepare the schedules, it is desirable that each property unit in each schedule be described so fully, and identified by such complete measurements, that no question could be raised as to its identification or as to the propriety of the use of the unit price that is subsequently assigned to it. It is entirely possible to prepare, based on field work alone, complete schedules of all visible physical property of a company operating any sort of a public utility, but it must be borne in mind that such schedules are not complete, because they ignore the history of the construction, and that by no means all the legitimate cost of creating or reproducing any property is represented by visible physical property, even where every item is capable of being fully measured. Careful consideration must be given to such history as shows any specially favorable or unfavorable conditions that may have been encountered during the construction of these items.

It must be emphasized that all items of material or work which entered into or were incidental to actual existing units are to be considered in the reproduced units as far as they can be determined.

Use of Records.—Most careful study of all existing records constitutes an essential part of the work of a reproduction estimate. Every plan, note, or cost record existing which relates to a plant unit measured and described in the field should be studied so that error may be eliminated and full data secured. It frequently occurs that plans are made for a structure and that during construction changes were made without notation on the original plans, such changes being indicated by fragmentary supplementary plans which are not preserved, or by no plans at all. In all such cases the appraisal record should correct the existing record as completely as practicable. Existing records should be used to the fullest extent to identify invisible items of property, such as underground construction or other inaccessible property, and to establish the nature and condition of existing items when originally installed.

Additional Information Not in Official Records.—Occasionally a case will be encountered where the history of the property is not to be determined with any degree of completeness from existing records, but on inquiry among former officials, former employees, consulting engi-

neers, contractors, or others who may have had connection with the property during all or a part of the building period, a large amount of thoroughly reliable information may be had which will throw great light on history and on costs. The use of all such data is highly desirable.

Little dependence should be placed on mere gossip or recollection unaided in any way by record, especially if it involves any increase in cost beyond a normal figure.

An illustration of the advantages to be gained from such an inquiry occurred in a recent valuation on a mid-western interurban railway. One of the branches of this road had been built as an independent line which changed owners once before coming into the hands of the present company. Aside from maps, a top-of-rail profile, and general data as to structures, all of which had been secured by a recent survey, the company had no records, not a word as to history, and nothing as to original cost. Inquiry developed the fact that all the construction records had been stored in the office of one of the officials, had not been turned over to the second owner, and after the death of this former official had been sold for old paper. Further inquiry yielded the names of one or two contractors. One of these men was visited. He went through his vault and unearthed complete profiles, copies of all progress and final estimates for grading, bridging, and track laying, plans for several bridges, and a large volume of construction photographs. With his aid, the men who purchased the right of way were found, and by their correspondence and receipts the original cost of most of the right of way (none of which was disclosed by deeds) was secured. This search disclosed the fact that many expenses were incurred which would not be even surmised by an examination of the property.

It is clearly the duty of the men engaged on an appraisal to take every precaution to secure to the fullest extent all data bearing on the actual work of construction as, in making an estimate of the cost of reproduction, due weight must be given to all evidence showing what the actual construction of the property included.

No Specific Assumptions as to Difficulties Where Not Capable of Proof.—In cases where no records are obtainable which will supplement actual field measurements and observed conditions, it would appear that the only proper course is to base the estimate of cost of reproduction on each unit of property as measured, assuming it to have been built under conditions which may be presumed from recent surroundings. To assume that the unit in question would cost less or more than the average unit, or that work has been omitted, or extra work done, in the case of a specific unit, when there are no records or obtainable data and nothing in the general property to justify the assumption, would be conjectural and carry little weight.

The Field Inspection.—The Committee deems it to be no part of its duty to undertake to outline methods in detail. It is proper to emphasize the necessity of full and complete records of field inspection in such form that they may be readily used in computation and fully indexed for filing. There is always a possibility of doubt regarding plans and records of old properties, uncertainty as to changes or abandonment, and, unless supported by field work, many really valuable records may be minimized in importance. It must be borne in mind that valuation work, especially the making of schedules, is of permanent value as a record of property, and that its value as a record for the Courts depends on the completeness of the description and identification of each property unit.

Many uncertainties can be cleared up, and much of the speculative character of estimating past conditions can be eliminated by good field work. There is a lack of significance, a failure to meet the requirements of appraiser or Court in every case where field inspection is hastily or carelessly made or no adequate record is made of the work.

Forms for Field and Office Record.—The use of forms is becoming so general and their value in securing uniform information from different field men is so great that there seems to be no ground for questioning their desirability or the propriety of their use. There are so many different forms required on a large appraisal and so many have been devised in connection with appraisal work in progress in the United States that no attempt is made to discuss the subject or to make suggestions. The adoption of full, complete, and carefully drawn field and office forms for field notes, schedules, and inventories is approved practice. The more completely all pertinent data regarding any class of property can be developed on a single set of forms without transcribing, the more satisfactory and convincing is the record.

Unit Prices on Physical Property Other Than Land.

There is no more important consideration in connection with valuation work than the correct determination of unit prices.

Necessarily, every large valuation involves a special study of the conditions which affect cost of work in the community and on the property, and of actual costs of various classes of work on the property under investigation. The prevailing rate of wages for different classes of labor, local prices on material, freight rates on material, or local contract prices, are by no means the only matters which are to be considered and taken into account.

Unit prices, or unit costs, which are finally determined upon and adopted as multipliers in the schedule should contain every element of cost which can be determined and allocated to each particular item.

A very erroneous result is likely to be secured if the appraiser adopts net cost for materials delivered to the property as shown by the vouchers for material, and actual contract figures for labor items, because in general these do not take into account all elements of cost to the owner.

In arriving at unit prices for final application, all relevant facts must be considered, and eventually sound judgment must be brought to bear on all data, if a proper unit is to be secured. The Committee desires to emphasize the fact that in the adoption of unit prices there is wide room for variation and error. A schedule may be made with the greatest of accuracy, field inspection may be most elaborate, and every possible check may be applied to the computations, but, if unit prices are carelessly adopted, which are too low or too high, a considerable error may be made easily in the valuation of tangible property.

The unit prices on most of the items entering into a valuation are not capable of exact and absolute determination or proof. This is especially true of all items in which labor largely enters into the cost.

The correctness of the figure finally adopted in every case depends in large measure on the reliability of records of actual costs, of similar units in recent years, the thoroughness of the study that has been made of those records, and the experience and soundness of judgment of the appraiser in the use of these studies and of other supporting data.

In the determination of unit prices the following general matters must be considered:

1.—What allowances should be made for contractor's profit in fixing prices?
2.—Shall unit prices be averaged over a period of time or shall actual prices as of the date of valuation be used?
3.—To what extent shall prices prevalent in piecemeal construction be considered?
4.—What should be included as elements of unit cost in fixing the price to be used?

1.—Contractor's Profit.—In assuming the reproduction of a property, the most reasonable assumption in general is that it would be let by contract, and that prices should be applied which would include the profit of contractors engaged on the work.

The fair total contract cost of the work, after award on fair competitive bidding, constitutes an equitable basis of unit cost determination, such cost including a reasonable profit to the contractor and necessary sub-contractors. To this should be added incidental unit

costs, overhead, administrative, and general costs incurred by the corporation, which are definitely assignable to the different units of property as part of the unit price.

There are cases, however, in which it would be a proper assumption that the company would establish its own organization, purchase its own plant and material, and do its own work.

The extent and character of the property under investigation, the actual history of the construction of the property itself, or the general practice in the part of the country in which it is located, will be the determining factors in arriving at the proper assumption to adopt.

When it is proper to assume that the company does the work itself, the estimated full cost to it should govern, including in the estimate proper recognition of the use of plant, hazards, and the fact that changes, modifications, and adaptation of the original plans generally occur and involve increased cost over the estimate, but no contractor's profit; unit prices should then be used, which represent what the estimated total unit cost to the company would be, by the method assumed to be proper.

2.—Shall Average Prices or Prices as of a Certain Date be Used? —When a valuation is made as of a certain date, it must be determined whether prices of materials and labor as of the date in question shall be used, or whether prices averaged over a period of years are more proper.

A study of the trend of prices of various commodities shows widely differing conditions. For example, Bessemer steel rail has remained steadily at $28 per ton, f. o. b. mills, since 1901 until recently; in some sections of the country lumber has increased with a fair degree of uniformity from 1901 to the present time, oak showing more than 50% increase, white pine more than 100% increase, and hemlock more than 50% increase; Portland cement, on the other hand, has decreased in some localities from $3 in 1880 to about $1 per bbl. in 1915, but increased in the year following; labor has steadily increased in recent years, with the upward trend still noticeable.

Other materials—an example of which is copper—have fluctuated, going up and down from extreme high to extreme low figures without any relation to changes in labor or other material prices. These facts compel most careful analyses of the records of the property under appraisal, and of other like properties in the same vicinity, to determine price tendencies, actual costs of work done, and special local conditions which affect costs, if average prices are to be used.

The illustration afforded by the Buffalo gas case (3 P. S. C., 2d Dist. N. Y., 1913), is given as showing the effect on valuation of price fluctuation. There were almost exactly 30 000 tons of cast-iron gas

pipe in question. The Commission summarizes the fluctuations as follows:

In 1897 when the property was taken over this would have been worth, at the prevailing price.	$547 500
Had a valuation been made in 1907, it would have been worth, at the price of that year.....	1 200 000
On the average prices of 5 years, 1893 to 1897....	617 700
On the average prices of 5 years, 1898 to 1902....	746 700
On the average prices of 5 years, 1903 to 1907....	944 400
On the average prices of 5 years, 1908 to 1912....	720 600

It seems to the Committee that what is desired is what may be termed normal present-day prices.

By normal, present-day prices is meant such prices, actual or average, as will be fairly representative of the period, but which do not reflect violent fluctuations up or down. These normal prices will be to some extent affected by changes in the market.

Many commodities, of which cement is an example, have had a downward trend, owing to increase in home manufacture and to improvements in the art. Other commodities, like lumber and timber of all kinds, have steadily increased in price owing to decreasing supply and greater demand. There can be no serious question as to the propriety of the use of normal, present prices, or prices averaged over 2 or 3 years at most, rather than the use of a long average which would bring the price above or below a proper, normal, present price.

In the case of such materials as iron pipe and copper, the price fluctuations of which have been violent during short periods, the problem becomes specially difficult. The actual construction of all large properties has extended over a period of years. An investigation of actual costs of construction on large properties will generally disclose the fact that materials have actually been bought at both high and low figures; frequently large quantities will be bought near one limit or the other. Actual cost, where ascertainable, gives actual investment in this material. Actual cost is not always capable of determination over the entire life of a property, and for such material as entered in during such period some arbitrary unit must be adopted, even in the case of "original cost" estimates.

The practice, adopted on some recent appraisals, of using a price derived from a weighted average of actual purchases over a period of from 5 to 10 years on the property under investigation, has the merit of using actual investment in recent years as a basis for determining a unit to be used on the entire property, and meets the objection raised by the Second District Commission of New York.

Present-day labor prices can be determined by an analysis of payrolls over such a length of time as will give proper actual averages

for each class of labor. By a comparison with similar data derived from records of other properties in the immediate vicinity, prices may be derived which are actual and are capable of proof. Prices thus determined would seem to be proper bases on which to build up estimates of total labor entering into the various units.

Any price which is used must be a matter of judgment, in the light of all available facts, but the arbitrary selection of a certain specific date as the date of appraisal does not seem to justify the use of prices which are abnormal. However derived, the prices used for fluctuating materials should be proper for the estimated period of reproduction which should end with the date as of which the investigation is made, and which should be sufficiently stable so that no reappraisal within a short time thereafter should make violent changes in estimates of reproduction cost.

3.—Piecemeal Construction.—Piecemeal construction has been defined to mean that kind of construction which is done piece by piece, and not at a single operation.

For example, the original pipe system of water-works and some important additions may be laid continuously in one construction period, either by forces of the company or by contract, but many additions must be made from time to time to meet the needs of an increasing population and a growing demand for water. These are necessarily short stretches of pipe, widely scattered, and are almost universally laid by the forces of the company at considerably higher costs than would prevail if the work could be done in connection with the building of an entire distribution system.

There are many kinds of construction, which always are built in what may be termed a piecemeal manner, which are not built at the time of original construction. Such work as the installing of water, electric, or telephone services, the building of industrial tracks and spur-tracks on the line of a railroad, the setting of meters and other forms of construction, is generally done in response to the demands of business development. Every effort of the management is bent to securing the greatest economy in doing this class of work; records are kept, and actual costs can be obtained over almost any period desired. Unit costs for this class of work can be determined from actual costs which are capable of very exact analysis and very definite proof by comparison with similar actual costs on other properties.

Whether or not unit costs should be determined on the basis of piecemeal construction, or on the basis of doing the work in a wholesale manner, depends upon the assumptions which are made with regard to the method of development of the property under which the estimate of reproduction cost is to be made. If on the basis of historic sequence, then the extra unit cost of piecemeal construction should be included; if on the basis of reproduction under present con-

ditions, during one construction period, then the extra unit cost of piecemeal construction should in general be excluded—the exceptions occur when it is assumed that the business is to be gradually developed in an assumed further period, in which cases it would be proper to use piecemeal unit prices for those items which would be so produced.

In the reproduction estimate, the property should be assumed as reproduced in the most economical manner and in reasonable sequence. Such assumption of necessity involves the building of some items of property that develop with the growth of business, not as part of the main property, but as following the main property, and at unit prices which would represent the actual cost of similar items of property built as they necessarily were or would be likely to be built by methods different from, and in some cases more expensive than, those that could be used on the main property.

4.—What Should be Included as Elements of Cost in the Unit Prices?—There has been an unfortunate lack of uniformity in valuation practice in the manner of arriving at the estimate of reproduction cost.

Apparently, some engineers have used material prices which were based on net costs at the point of delivery, and labor costs based on contract prices for performing certain work, and then, realizing that these do not give complete cost, but without making a full analysis, have added percentages to the total valuation for omissions, contingencies, incidentals, errors, contractors' profit, or other blanket items.

It may be that such a method, when used with good judgment, will lead to a result which is as close to the truth as when a more elaborate analysis is made, but it is much more difficult of satisfactory proof.

In the opinion of the Committee, the unit prices which are applied to specific items of property should include every element of cost to the owner capable of definite assignment to those items, and no such elements should be included as a blanket, overhead charge.

For example, the material may be purchased in the open market, the normal price may be capable of determination, and the freight rate to the point of use may be secured and applied, but to these two main elements there should be added, wherever they are properly applicable and obtainable:

(*a*)—The cost of purchasing;
(*b*)—House or material yard service;
(*c*)—Local transportation;
(*d*)—Waste, loss, and breakage;
(*e*)—Direct supervision.

(*a*)—*The Cost of Purchasing.*—The organization and expense of the purchasing department might with propriety be treated as an over-

head charge; it may with equal propriety be treated as a charge to material, and is capable of determination with definiteness in most cases. In the case of a large operating company, where purchases cover material for maintenance and also large expenditures for construction, there is grave danger of overlooking this item of expense. Usually an analysis of the accounts will give actual costs of purchasing of large quantities of material, thus permitting an allocation of purchasing costs on construction material.

(*b*)—*House Service or Material Yard Service.*—The cost of warehousing, material yard handling, or what may be termed "House Service" or "Material Yard Service", is an expense which on a large property is great in the aggregate although a comparatively small percentage on cost of material purchased. The extent of the charge and its division depends on how much of the material is purchased for direct delivery, how much goes through a material or storage yard, and how much goes into a warehouse for delivery in small lots as needed.

(*c*)—*Transportation.*—In almost all valuations, the greater part of the material involved incurs a transportation expense over and beyond the railroad freight rate to the point of delivery. In the case of railroad materials of all kinds, the material is usually purchased, f. o. b. cars, at the most convenient delivery point on the line of the road, and a haul of possibly several hundred miles on the company's own line is necessary. The determination of the average length of haul for the material of each class on a railroad and the determination of a proper rate to apply constitute a very important element in the determination of the correct unit prices. In connection with this matter of the rate, attention may be called to the fact that, under construction conditions, on a new road, with traffic all in one direction, without the advantage of an old established operating organization, the cost will exceed the cost of service on an old road. This has been recognized by the Oklahoma and California Commissions.

The local transportation may consist of haul by truck or wagon to the site, and, if so, is a present element to be computed, if not otherwise provided for. In estimating cost of transportation, it is to be assumed that all transportation facilities existing at the date of estimate—except the property under consideration in case it is a transportation property—are available.

(*d*)—*Waste, Loss and Breakage, or Overplus Over Measurements.*—These are elements of either unit cost or of allowance in measurement of quantities rather than of contingencies, for the reason that they vary so greatly, from nothing at all on some materials to a very considerable percentage on others. In this class would come allowances for sag in wire or cable, waste in lumber, loss and breakage in fragile materials, and all other allowances over net quantities which are necessary to in-

sure enough material to complete the work. In the case of some materials, quantities are derived by rules which give theoretical or geometric quantities, though the actual quantities moved and paid for may considerably overrun.

One very notable example of this is given in the Panama Canal Report for the year ended June 30th, 1912, which states that the geometrical quantity in the dredged channel of the Panama Canal, from deep water on the Atlantic side to a short distance beyond the shore line, was to that date 10 169 000 cu. yd., though the actual quantity taken out of the channel amounted to 22 886 000 cu. yd., an excess of 125 per cent. The great excess of dredging in this case over the geometrical quantity was due to the quantity of material washed into the channel by waves and currents.

There are certain standard weights of cast-iron water pipes, and pipes of these weights are ordered. As the pipe foundries cannot make pipes of the exact weight, the full price per ton is paid for any additional weight that there may be, up to a certain limit. The pipes are not allowed to run much below the standard, because it is provided that the thickness of the metal shall not be less than the prescribed thickness at any place by more than a very limited amount. The average actual weights, therefore, are more than the standard weights.

In valuing an earthen dam, the net volume in the embankment is generally measured or computed from the records, and it might be correct to use this net volume if the unit prices to be used were derived from the cost per cubic yard of other similar embankments. If, however, the unit prices to be used were derived from measurements of the quantity of material taken from borrow-pits for the purpose of building the dam, a considerable allowance should be made for the shrinkage of the material when placed in the embankment and consolidated, and for the waste of material which almost inevitably takes place in building a dam across a stream as the result of floods and other causes. The waste, as a rule, is exceptionally large when a dam is built by the hydraulic fill process.

It seems much better that judgment should be used in determining the actual quantities of material involved in a given construction than to make an allowance for such additional material as a contingency.

(*e*)—*Direct Supervision.—Inspection.*—It has been the custom in many appraisals to treat as an overhead charge, under the account "Engineering" all charges for Engineering, Inspection, and Supervision of work, and to base such charges on a general percentage of the entire physical property estimate.

The Committee holds the opinion that, wherever it is possible to do so, charges for direct supervision should attach to the different units or groups, rather than to the property as a whole, and that all

such charges as can be localized to certain materials, units, or groups should be treated as affecting the unit price, and should not be added as a percentage. This would permit the charge for general engineering to be treated as an "Overhead Charge" and include only those costs of general engineering, surveying, design, and supervision which were applicable to the property as a whole, and not capable of definite assignment to different groups of units or individual units of property.

As examples of elements of cost properly assignable to units or groups of units, the Committee cites the following cases:

The inspection cost on rail, water pipe, structural steel, and other materials purchased is capable of separation and inclusion in the unit price.

The local engineering and inspection costs of a large building, a dam, an underground conduit line, or other individual units or groups of units may be capable of analysis and comparison with like costs incurred in the construction of other like units, and a more accurate estimate is likely to be secured than if the total engineering charges be found as a percentage of the cost of the entire property, based on the experience with similar properties.

Each valuation must of necessity be an independent work, and each unit price should have such consideration as will insure that it is applicable to the case in hand, and that all elements, of which those above stated are only a part, have been taken into account. When unit prices for different items in the schedule are made up on the basis of the amount of labor involved, it must be remembered that the labor costs are by no means covered by the sums included for laborers on the pay-roll. There must be, in addition, proper allowances for insurance, injuries, and damages, lost time of monthly men, costs or losses due to interruption, or failure to co-ordinate the work at all times, decreases in efficiency due to such conditions, the cost of supervision, including accounting and timekeeping, and other actual costs.

When unit prices are based on contractor's prices, most of the items above mentioned would naturally be included by the contractor in such prices or as extra work, for which payment must be made to the contractor; but, wherever contract work is done, there is generally a considerable additional cost to the company as the result of subsequent work by its own forces.

Illustrations of Actual Cost Affecting the Cost per Unit.—One illustration which shows the extent of these items that have been referred to, and the desirability of more detailed analysis of costs than is frequently made, is that of an electric light and power property in the Middle West, in a city of more than 400 000 population.

During a period of 2½ years, the company spent in construction $9 476 000, divided as follows:

Actual material entering into permanent construction.		$6 808 000
Actual labor costs in building permanent work.....		1 515 000
Total cost of labor and material in structures capable of scheduling.....		$8 323 000
A.—Construction tools.....	$110 000	
B.—Store and supply expense.....	288 000	
C.—Purchasing expense.....	39 000	
D.—Injuries and damages.....	48 000	
E.—Engineering, general.....	122 000	
F.—Engineering, direct supervision.....	546 000	1 153 000
		$9 476 000

This does not take into account costs for interest, taxes, organization, or other costs which were incurred in the nature of actual overhead costs.

The aggregate cost, over and above those of bare labor and material, was 12.1% of the total, or 13.8% of the items capable of inventory.

It was found that full analysis of these costs gave very different percentages for different classes of construction, as for example:

Tool expense was approximately 2% of labor and material on overhead and underground construction, and 1% on buildings and equipment.

Stores and supplies expense, or the cost of handling, warehousing, and material yard, varied even more widely, as follows, being based on total material in each account:

Buildings	2.78%
Steam plant equipment.....	0.74%
Electric plant equipment.....	5.96%
Lines overhead and underground.....	7.35%
Other property.....	2.45%

Direct supervision, or those engineering and inspection charges which were capable of exact assignment, bore the following relation to total labor and material charges to each account:

Buildings	8.46%
Steam and electric plant equipment.....	7.36%
Lines overhead and underground.....	4.60%
Other property.....	1.37%

These varying percentages are a strong argument in favor of more careful cost keeping on construction work and of more careful analysis

of existing records of actual cost, when a valuation of property is made.

Illustrations of Earthwork Unit Costs.—There are various classes of construction, such as earthwork, masonry, concrete, and other like construction, in which labor largely enters, where the conditions of construction may vary widely, and where there are available many contract prices, published or unpublished. The danger of basing a series of unit prices on contract prices, without analysis of further actual costs, may be shown by the discussion of one of these, and earthwork appears to offer a number of excellent examples.

In building a railroad or other property involving large quantities of earthwork, it is usual to pay for material, according to classification, as earth, loose rock, solid rock, etc. It is also usual to pay for overhaul beyond a certain specified haul limit.

In making estimates of cost of reproduction, unless all original notes, profiles, and estimates are available, it is a difficult and expensive matter to secure even a close approximation of actual yardage, and practically impossible to determine classification of loose rock, hardpan, or other material the character of which changes due to the effect of weather, or the identity of which may be concealed by growth of vegetation or otherwise. It is equally difficult to determine the quantity of overhaul or the amount of work which may have been done as a necessary incident to the grading of the embankment, such as changing of stream channels or roads.

All these things were encountered in the building of the property, and must in some way be accounted for in its reproduction. In some cases the unit price on unclassified earth in place is the fairest figure, and capable of better support than would be the attempt to classify or to separate overhaul.

In addition to these elements of cost which may be encountered in all earthwork, the railroad company encounters the expense to the company of transportation of men and contractor's plant, and the cost of work done after the contractor leaves the job.

The adopted unit price should be such a figure as will cover all expense, of every nature whatsoever, as is directly chargeable to grading the measurable embankment. Three or four examples will illustrate the possibility of grave error in failure to analyze unit costs sufficiently:

Example 1.—A high-class interurban railroad built in Michigan during 1912-13-14.—

Total material moved.....924 630 cu. yd.
Contract price of overhaul.....0.01 per yd. station.
Hardpan, if encountered, to be paid by agreement.

In this case the company actually paid on its final estimate for:

774 387	cu. yd. earth	at $0.28	$216 828.36
1 860	" " loose rock	" 0.40	744.00
300	" " solid rock	" 1.00	300.00
776 547	Total classified material at prices named in the contract.....		$217 872.36
148 083	In addition to which, there was allowed: cu. yd. "Hardpan" and "Wet Earth" at 38, 40, and 50 cents.....		60 912.94
924 630	Total cu. yd., unclassified, at $0.30, average.		$278 785.30

It thus appears that the "rock" is a negligible item, but that the average cost of the unclassified material exceeded the contract price of earth by 2 cents per cubic yard.

In addition, the company paid:

For overhaul.....	$29 237.51
For force account on work after contractors left...	13 988.42
Total.....	$43 255.93

Thus making the total direct cost of the work $322 011.23, or an average of $0.348 per cu. yd.

Practically all of this excess was for elements of cost which could not be determined from either a careful reinspection of the property or from most accurate measurement, and, if reflected in the reproduction estimate at all, must appear in the unit price.

Example 2.—Among the exhibits introduced in the Minnesota rate case was one by A. H. Hogeland, M. Am. Soc. C. E., Chief Engineer, Great Northern Railway, entitled "Hogeland, 2d Amended Exhibit 4", which gives in detail the cost of grading on lines built between 1898 and 1910 aggregating 1 553.2 miles. This exhibit shows the contract prices for each class of material, and also the total charges to grading. The exhibit shows the details for each line built during those years.

A very brief abstract of this exhibit shows:

Earth...	28 693 523	cu. yd.	average contract price 19.3 cents	$5 545 074.31
Hardpan	9 260 156	" "	" " 34.7 "	3 211 293.53
Loose rock..	3 115 093	" "	" " 40.7 "	1 270 666.25
Solid rock..	6 209 349	" "	" " 94.7 "	5 876 720.69
	47 278 121	" "	average of all..... 33.6 cents	$15 903 754.78

Brought forward	$15 903 754.78
It is to be noted that, over and above these contract prices, the company paid for overhaul, force account items, transportation, and other such charges, the total sum of	4 928 725.25
Total cost	$20 832 410.03
Average cost per cubic yard	44.11 cents
Cost in excess of contract price	10.5 cents

These figures are affected by the large yardage of rock, which presumably could be largely determined in a valuation. In order that rock be eliminated, an analysis of this exhibit was made and 300.29 miles having no rock were tabulated separately.

On this mileage the total yardage moved was 3 293 244 cu. yd.
Total contract price, $506 869, an average of 15.4 cents per cu. yd.
Total Company cost over contract, $264 727, or 8 cents per cu. yd.
A total cost of $771 596, or 23 cents per cu. yd.

It is here shown that actual cost per unit exceeded contract price per unit by 52.22 per cent.

Example 3.—A steam railroad built in Wisconsin in 1912 and 1913:

Total contract price for grading		$1 259 567.81
Extras		
Contractors extra bills	$42 830.86	
Railway Company labor	8 413.74	
Train service	215.03	
Rental of equipment	522.79	
Miscellaneous material and labor	2 028.98	
Transportation of contractors men	46 736.96	
Transportation of equipment	73 483.36	174 267.72

Percentage of extras to contract price 13.84 per cent.

In this contract there were the following quantities:

Earth excavation, 4 097 881 cu. yd. at 20 and 22 cents	$868 314.66
236 300 " " classified material	137 589.38
4 334 181 cu. yd., at average 23.2 cents	$1 005 904.04
Added to this was overhaul	253 663.77
	$1 259 567.81

Thus the overhaul added to the cost 6 cents per cu. yd., or 25 per cent.

"Example 4.—In the construction of the Musconetcong tunnel of the Lehigh Valley Railroad, the contract cost, based on unit prices,

a list of which is given in Drinker's 'Tunneling' (page 1085), was $883 938, whereas the actual final cost, including the payment of contractors' claims not settled until four years after the completion of the tunnel, was $1 282 149, an increase of nearly 50% over the cost as reflected in the commonly accepted unit prices."

These few examples indicate clearly the danger of adopting arbitrarily, as a unit price for reproduction cost, some figure based on a contract price or an average of contract prices, without most thorough study of the results of actual similar work. Many other illustrations as to the effect of the several elements of cost on the final price of various classes of work into which labor largely enters, and in which the hazards of construction are great, might be cited. Enough has been said, however, in the opinion of the Committee, to emphasize the statement that in each case conditions must be examined to find out what elements of cost do enter into the unit price, and that each unit price must be supported, if supported at all, by actual costs of similar work under similar conditions.

Unit costs found by general inquiry of supplying companies, engineers, contractors, and others are rarely reliable because of the lack of full knowledge on the part of those furnishing the information concerning the character of the work and the specifications and conditions under which it is to be done, and because of the fact that they are not likely to make the detailed, intelligent study which might justify subsequent expression of opinion. Even competitive bids on detailed specifications may not furnish reliable figures. Recent actual final construction costs on the plant being estimated or on similar properties, located under similar conditions, furnish the safest criteria. All such data should be interpreted in the light of sound judgment and wide construction experience.

An examination of the tabulated bids of any large contract letting will disclose the unreliability of individual opinion. It must be borne in mind that such bids are based on identical plans and specifications and conditions. It is not at all unusual to find variations in bids on individual items varying from 15 to 40% to as much as 100% or more. In some cases the opinions of contractors, as expressed by their bids, may represent a proper figure based on wide experience and full knowledge of local conditions, but other bids may be in no wise representative, and be radically high or low.

To conclude the discussion of unit prices, the Committee is of the opinion that a rational sequence of construction should be followed in the reproduction of a property; and that rational assumptions should be made as to the manner of doing the different parts of the work, whether by the forces of the company or by contract. Unit prices, based where possible on the actual cost of doing similar work, in a

similar manner, under similar circumstances, should be determined by persons of experience and sound judgment.

If, on the basis of the assumed conditions, piecemeal construction is required, and if the most appropriate way of executing such piecemeal construction is by doing the work directly by the company's forces within the assumed periods of construction and development, then the cost of work done in this way should be used in determining the reproduction cost. If the assumed conditions indicate that other portions of the work could be done more economically by contract, then the unit prices should have as their basis the price which a contractor would charge for doing such work, but to this must be added all other elements of cost or expense to the owner which can be separately allocated.

The unit prices should be based on the normal average cost of work for a considerable period—say, 5 or 10 years—in order to give stability to the valuation, so that it may be used for a subsequent term of years. In the case of items which are steadily increasing or decreasing in value, the prices adopted should be normal for the time of the valuation.

Full consideration should be given to the time allowed for construction, and due weight given to climatic conditions and the effect of weather on the cost of the work.

The Committee emphasizes the necessity for full information from trustworthy sources as to the actual complete cost of doing similar work under similar circumstances. The greater the amount of such data, making possible the adoption of complete and correct unit costs, the smaller the amount of overhead percentages which are necessary for providing for proper total construction costs.

The Cost of Reproducing Land Holdings.

For the reproduction cost of land holdings the Committee will base its arguments and conclusions on the law as expressed by selected and apparently authoritative Court decisions, and on those results of attempts of public or private purchasers to acquire properties for public purposes which are of common knowledge to engineers and others having to do with such matters. Examples of some of these results, fortifying the arguments and conclusions of the Committee, will be given.

Lands not All of Similar Character.—Classes of Land.—Not all lands acquired by public service corporations fall under one class; there are radical differences, and it is essential that a proper classification of them be made in any valuation.

Three classes result from a consideration of the extent and character of the acquisition:

1.—The acquisition of a single whole parcel, which may be selected from among several available parcels.

2.—The acquisition of a number of whole contiguous parcels to form a tract of moderate size.

3.—The acquisition of a number of parcels, both whole parcels and parts of parcels, to form a continuous strip, such as a railroad right of way, or a large body, like a collecting or storage reservoir.

1.—Single or Detached Parcels of Land.—In many instances lands are acquired and used by public service companies, which are in detached parcels of ordinary size, not connected with the other lands owned or used in the conduct of the business. These lands are not acquired in response to a need for the particular parcel. Any parcel of the same or similar area, in the same general locality, would meet the requirement of the company nearly or quite as well. Examples of this class are seen in lands acquired for electric generating- or sub-stations, car barns, warehouses or material yards, gas plants or gas-holders, office buildings, and other similar uses.

It may be that location on a water-front, or a point of convenient access to side-track facilities, is a desideratum, but any one of several parcels, which meets those requirements in the vicinity, may serve the purpose. It would seem to be clear that in such cases the same general laws which regulate values of land for any other purpose in the immediate neighborhood would govern, and that the reproduction cost of the land would be fixed by the value of other lands used for general purposes, as shown by sales, asking prices, or in any other manner that is usual and proper.

Presumably, the public service corporation, in the same way as an individual seeking a factory site or residence, would have a fairly wide range of selection and would choose that tract which would meet the requirements and could be had at the most reasonable price. It is fair to assume that such purchases would be at substantially the market value of lands in the vicinity at the time of acquisition, and would be subject to the same increase or decrease in value as other near-by similar lands the values of which would be affected by the same causes.

Therefore, it would appear that the reproduction cost of such a holding as an ordinary lot, or a tract of not unusual size, which is entirely disconnected from other holdings, should be based on adjacent or near-by land values for general purposes, should include no severance damages, but should include a reasonable allowance for the costs of acquisition.

2.—Tracts of Land.—Tracts of land of considerable area, including a number of lots or blocks, offer a more difficult case, for, aside from any such expenses as vacation of streets or alleys, or other work done to bring the tract as a whole into a usable condition, there is likely to have been originally an element of enhanced cost, due to the fact that the demand for a large number of contiguous small parcels tends to enhance the price rapidly, and even in a district where a large part of the lands are for sale, the demand affects the price before all can be acquired. Usually, in the acquisition of such a body of land, made up of many small parcels, some owners will have to be dealt with who are unwilling sellers and who hold out for an extreme price, making necessary either condemnation, or the forced payment of a higher consideration than would ordinarily be the case. Any condition necessitating the purchase of a number of adjacent parcels and uniting them into one large tract suitable for a specific purpose tends to remove the transaction from the simpler first case and take away the free choice of sites. Such cases demand a much more careful study of costs, especially with reference to recent accumulation of property of the same kind.

It may be seen readily, for example, that the purchase of 50 or 60 lots from numerous owners may increase the price per lot far above the normal price under a demand for only one lot or a few lots. The immediate demand for all the lots may and usually does add materially to the cost of the whole. Later, the demand being supplied and ceasing, prices drop to the old level, so that a valuation based on later sales of neighboring lots would show less than original cost and less than any possible cost of reproduction at the time of making the estimate.

Although general land values will have an important bearing on the price, consideration based on actual experience in comparable situations should be given to the effect of the large and pressing demand.

It is argued that, in such a case as the one cited, actual original cost, and not reproduction cost based on present land values, would be ample allowance, especially as in many cases the actual original cost of the assembled tract would be considerably in excess of the present value of neighboring lots for general purposes. It would seem to be fair, however, in those cases in which the original cost of an entire assembled tract exceeded the total market value of all the separate parcels at the time the acquisition was commenced, to assume that, if general values of lots increase in the vicinity, the cost of reproduction of the tract acquired for a specific purpose should, as a general rule, increase *pro rata*.

Similar questions are raised where lands are acquired for the extension of an existing plant. In such cases, prices may be and sometimes are demanded and necessarily paid which are far above normal.

In the cases cited there may be some freedom of choice as to the exact lands, but the difficulties due to acquisition of numerous parcels will be encountered no matter which of several possible tracts is selected. The reproduction estimate should be based on the value of land in the vicinity, the figure adopted including the incidental costs of acquisition of private titles, severance damages, if there was any severance, and the cost of acquiring any rights requisite for the use of the tract as a whole, such as vacation of streets or alleys, and the purchase of leases. All recorded excess costs above a normal market value should be given due weight. Only thus will the present reasonable cost of reproduction be obtained.

3.—Connected Strips or Bodies of Land.—By far the greatest difficulty arises when required lands constitute a connected strip or body, and there is no option in the choice of lands, all of which must be acquired to form the required strip or body.

An example of this class is furnished by a steam or electric railroad right of way. Here, the lands are a narrow strip, not purchased in ordinary commercial units of 10, 20, or 40-acre tracts, or of lots or other subdivisions of a plat, but in small fractions of such tracts taken without reference to the effect on the remainder of the tract from which they were taken. The location is determined by topography and continuity of route, and may not be varied on account of the objections of a land-owner. All the right of way must be acquired.

Another example is that of flowage lands for a reservoir. In this instance, many entire parcels must be acquired, and unless a greatly excessive acreage is taken, many irregular fractions must be bought. Here, as in the railroad right of way, conditions of topography and continuity govern. It must be this particular land and no other, and all of it must be had.

All valuations of lands acquired for any use where all must be had, and no choice or option as to the selection is left the purchaser, must rest on the same general principles of valuation. The Courts have clearly established the general principles which govern the fixing of value of property condemned for public use.

All elements of value must be considered. The owner must be recompensed, not only for the value of the land taken, but for all damages or losses of value suffered by him on account of the taking.

Under the law interpretation of just compensation, there can be no doubt that in cases of water-works reservoirs, railroad rights of way, flowage lands, or other properties of like nature, where the lands are not necessarily in uniform parcels, and where part of the land is taken and part cut off and left, just compensation will include the value of the lands taken and the money equivalent of any damages suffered by the owner by reason of the taking. This is all value to the seller and represents what is taken from him.

It would seem that these same principles are applicable in the valuation of such property in making a reproduction estimate.

The Term "Just Compensation" as Defined by the Courts.—The Constitution prescribes that private property shall not be taken without just compensation. The rules governing the condemnation of private property have been thoroughly established by a long line of decisions. All elements of value must be considered. The owner must be recompensed, not only for the value of the land taken, but for all damages or losses of value suffered by him on account of the taking.

The terms, "compensation", "just compensation", "due compensation", etc., have been defined in many cases.

In Virginia & Truckee R. R. Co. *vs.* Henry, the Court said (8 Nev., 165):

"It is difficult to imagine an unjust compensation; but the word 'just' is used evidently to intensify the meaning of the word 'compensation'; to convey the idea that the equivalent to be rendered for property taken shall be real, substantial, full, ample; and no legislature can diminish by one jot the rotund expression of the constitution. So are all the decided cases. While courts have differed upon minor points, two of which the statute of this state settles, namely, the allowance for particular benefit derived from the construction of the railroad and the exclusion from the calculation of damages of the cost of necessary cattle guards and fences when the petitioner offers to construct, yet upon the great substantial underlying basis upon which only can arise a constitutional law for the taking of private property for public use—the absolute protection of the individual by just compensation—there has been, could be, no dispute."

In Pick *v.* Rubicon Hydraulic Co. (27 Wis., 445), the Court said:

"In the case of Bigelow *vs.* The Western Wisconsin Railway Co., decided by this court at the present term, it was held that 'just compensation' consists in making the owner good, by an equivalent in money, for the loss he sustains in the value of his property by being deprived of a portion of it; and that it includes not only the value of the portion taken, but also the dimunition in value of that from which it is severed. In establishing this rule, we followed the reasoning of this court in Robbins *v.* Milwaukee and Horican R. Co., 6 Wis., 636; in Snyder *vs.* Western Union R. Co., 25 Wis., 60; and in Welch *v.* Milwaukee and St. Paul R. Co., 27 Wis., 108; and adopted the doctrine of numerous adjudications by the courts of other states upon the same subject."

The cases cited show the general rule as to the taking of lands by condemnation. There can be no doubt that, in the case of a waterworks reservoir, a railroad, flowage lands, or other property of like nature, where the lands are not necessarily in uniform parcels, and where part of the land is taken and part cut off and left, any amount necessarily paid as just compensation or as damages is a cost which

cannot be evaded by the purchaser. It is value to the seller, reimbursing him for what has been taken from him. It is an actual cost to the purchaser which is properly chargeable to the capital account.

The reproduction cost, or present cost of acquisition, does not include elements of value arising subsequently to and because of the taking for utility purposes. It includes only such sums, for the land itself, as the several owners would be entitled to in case of present acquisition, plus costs of purchasing.

Same Elements Must be Considered in Valuation as in Fixing "Just Compensation."—It would seem to be clear from a study of all the cases that, in valuation of lands which partake of the nature of rights of way or flowage lands, the same principles should govern as in condemnation, and all such elements should be considered in valuation as are taken into account in fixing just compensation in condemnation. Thus, the cost to the utility at the time of acquisition is not to be substituted for, or taken to be, the equivalent of reproduction cost, because the land may have declined or increased in value since the acquisition. Nor does the source of the title, or of the funds used to acquire, affect the matter, because that which is paid for out of surplus, or that which is donated, is as much the property of the company owner as that acquired by the expenditure of other capital and payment of full value.

The market value of the land actually taken from an owner is one element of cost, the damage to remaining and adjacent lands of the same owner is another, the costs of all investigations of title, attorneys' or agents' fees, surveys, plans, and expenses, and all other costs in connection with the transaction, constitute another. All taken together make up the price of the property acquired, and, as a general rule, in the absence of mistakes, dishonesty, or extravagance, may be taken as the value at the time of investigation. As said in the Minnesota rate cases (page 451):

"It [the utility] would be free to stand upon its legal rights and it cannot be supposed that they would be disregarded."

Thus, the presumption is that the amounts in condemnation judgments are not excessive. And, as a general rule, in the case of acquisition by purchase, it may be assumed that the utility does not pay more than the owner is legally entitled to receive.

All these elements of value for which "just compensation" would be rendered in a condemnation, exist in greater or less degree and must be considered, whether the land was actually acquired by condemnation or by private purchase. It occasionally happens in the case of many corporations not having the right of eminent domain, such as hydro-electric power properties in some States, that the prices paid are excessive of necessity.

Market Value of Adjacent Lands, per Acre, Not Always Conclusive.—It must be remembered that general land values are based on sales where a willing seller deals with a willing buyer. In the case of farm lands, for example, a man who desires a farm in a certain locality has the choice between a number of pieces that are on the market. He may be satisfied with one 40-acre farm, but considers the price too high. He has the option of paying the price or taking another farm at a lower price. As has been said, the railroad or the water company has no such option, but must take that parcel and no other. Experience has shown that condemnation juries often fix a high price and that often it is better business judgment to pay as large a sum as the condemnation prices, in order to avoid the expenses of condemnation, the delay caused by the proceeding, and the feeling aroused in a community by litigation. It often occurs that contiguous lands have not the same availability or value as the lands acquired by a utility, and only when they are comparable can the market value of such lands be used as a basis for estimate.

From the nature of the strips of land acquired for public utilities, the compensation paid, when measured in terms of dollars per acre on lands actually taken, appears to be very high compared with the per acre price paid in sales of similar lands for general purposes.

For example, a railroad company condemned a strip of land, 100 ft. wide, across a tract containing approximately 40 acres, cutting it in two nearly in the middle. The strip taken was 1 600 ft. long and contained 3.65 acres, leaving 36.25 acres to the owner, divided approximately 20 acres on one side of the road and 16 on the other. The building of the railroad necessitated changing fields and drainage, introduced two farm crossings with gates to open and shut in perpetuity, and not only cut off a corner of an orchard, but introduced a fire hazard, as the stables were near the line.

All these considerations caused a jury to fix as compensation the sum of $100 per acre for land taken, which was a fair figure based on several recent sales, and the further allowance of $15 per acre as the damages to the remainder. The compensation, therefore, was, for land, $365, for damages, $545, a total of $910, to which was added acquisition and overhead costs in excess of $170, or a total cost to the company, exclusive of interest, of $1 080, or almost exactly three times the acreage price fixed by the jury.

Several purchases at private sale in the same county were at approximately the same figure.

In some of the early valuations, notably the State railroad valuations in Michigan, Wisconsin, and Minnesota, the attempt was made to establish the relation between sales to railroads and sales to individuals by a study of the record of actual sales. Based on these

studies, a system for fixing the cost of reproduction of railroad lands by the use of multiples applied to farm land values was adopted.*

The Michigan tax cases, in which this method of valuation was used, reached the Courts without any contention being raised as to the methods or results of the valuation, but on other issues. Hence, the question was not passed on. The Minnesota valuation of land by this method became an issue in the Minnesota rate cases, and the method was not approved by the Supreme Court.

The Minnesota Rate Case Decision.—In many respects the most important decision bearing on land valuation is that of Mr. Justice Hughes in the Minnesota rate cases. This decision is of such importance, and has had so many interpretations placed on it, that extended quotation from it appears to be desirable. The decision must be construed as a whole, and all parts of it bearing on the subject of land must be studied together, in arriving at a conclusion as to the exact meaning. (Simpson *et al.*, constituting the Railroad and Warehouse Commission of the State of Minnesota *v.* Shepard, 230 U. S., 352, *et seq.* (1912), Syllabus, page 355.)

"For fixing rates the basis of calculation of value is the fair value of the property of the carrier used for the convenience of the public." (Smyth *v.* Ames, 169 U. S., 466.)

"There is no formula for the ascertainment of the fair value of property used for convenience of the public, but there must be a reasonable judgment having its basis in a proper consideration of all relevant facts. * * *

"Assets and property of a carrier not used in the transportation business cannot be included in the valuation as a basis for rate-making.

"Property of a railroad company cannot be valued for a basis of rate-making at a price above other similar property solely by reason of the fact that it is used as a railroad, and increases in value over cost cannot be allowed beyond the normal increase of other similar property."

After setting forth the values allowed by the Master and the Court below for land in the Northern Pacific case, stating the contentions in the case, and quoting from Mr. Cooper's testimony at some length, in order to show how he arrived at his estimates of value, and after bringing out the fact that "market value", as used by Mr. Cooper, was intended to mean, not the value of land for general purposes, but what it would cost the railroad to acquire the land, the Court says:

1.† "In reviewing the findings, the court below reached the conclusion that 'the master in effect found that the cost of reproduction

* "The Valuation of Public Service Corporation Property", by Henry E. Riggs, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXII, pp. 59–64, inclusive. Discussion by the late W. D. Taylor, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LII, p. 359. Report of Mr. Dwight C. Morgan to Minnesota Railroad and Warehouse Commission.

† Paragraphs are numbered by the Committee for convenience in referring to them in discussion.

and the present value of the lands for the terminals in the three great cities, including therein all cost of acquisition, consequential damages, and value for railroad use which he allowed, was only about 30 per cent. more than the normal value of the lands in sales between private parties. He found the value of the lands outside the terminals to be only twice their normal value.'

2. "From our examination of the evidence we are unable to conclude that the excess stated may be thus limited. What is termed the normal value does not satisfactorily appear. It further will be observed —from the summary of valuations we have set forth in the margin— that the amount thus allowed in item 1 for lands, yards, and terminals, both in and out of the three cities ($21,024,562), was included in the total on which 4½ per cent. was allowed in item 30 for 'engineering, superintendence, legal expenses', and again was included in the total on which 5 per cent. was allowed in item 37 for 'contingencies', and, in addition, was included in the total on which 10 per cent. was allowed in item 39 for 'interest during construction'.

3. "These are the results of the endeavor to apply the cost-of-reproduction method in determining the value of the right of way. It is at once apparent that, so far as the estimate rests upon a supposed compulsory feature of the acquisition, it cannot be sustained. It is said that the company would be compelled to pay more than what is the normal market value of property in transactions between private parties; that it would lack the freedom they enjoy, and, in view of its needs, it would have to give a higher price. It is also said that this price would be in excess of the present market value of contiguous or similarly situated property. It might well be asked, who shall describe the conditions that would exist, or the exigencies of the hypothetical owners of the property, on the assumption that the railroad were removed? But, aside from this, it is impossible to assume, in making a judicial finding of what it would cost to acquire the property, that the company would be compelled to pay more than its fair market value. It is equipped with the governmental power of eminent domain. In view of its public purpose, it has been granted this privilege in order to prevent advantage being taken of its necessities. It would be free to stand upon its legal rights and it cannot be supposed that they would be disregarded.

4. "It is urged that in this view the company would be bound to pay the 'railroad value' of the property. But supposing the railroad to be obliterated and the lands to be held by others, the owner of each parcel would be entitled to receive on its condemnation its fair market value for all its available uses and purposes. (United States *v.* Chandler-Dunbar Water Power Co., decided May 26, 1913, 229 U. S., 53.) If, in the case of any such owner his property had a peculiar value or special adaptation for railroad purposes, that would be an element to be considered. (Mississippi, etc., Boom Company *v.* Patterson, 98 U. S., 403; Shoemaker *v.* United States, 147 U. S., 282; United States *v.* Chandler-Dunbar Co., *supra.*) But still the inquiry would be as to the fair market value of the property—as to what the owner had lost, and not what the taker had gained. (Boston Chamber of Commerce *v.* Boston, 217 U. S., 189, 195.) The owner

would not be entitled to demand payment of the amount which the property might be deemed worth to the company, or of an enhanced value by virtue of the purpose for which it was taken, or of an increase over its fair market value, by reason of any added value supposed to result from its combination with tracts acquired from others, so as to make it a part of a continuous railroad right of way held in one ownership. (United States *v.* Chandler-Dunbar Co., *supra*; Boston Chamber of Commerce *v.* Boston, *supra.*) There is no evidence before us from which the amount which would properly be allowable in such condemnation proceedings can be ascertained.

5. "Moreover, it is manifest that an attempt to estimate what would be the actual cost of acquiring the right of way, if the railroad were not there, is to indulge in mere speculation. The railroad has long been established; to it have been linked the activities of agriculture, industry, and trade. Communities have long been dependent upon its service, and their growth and development have been conditioned upon the facilities it has provided. The uses of property in the communities which it serves are to a large degree determined by it. The values of property along its line largely depend upon its existence. It is an integral part of the communal life. The assumption of its non-existence, and at the same time that the values that rest upon it remain unchanged, is impossible and cannot be entertained. The conditions of ownership of the property and the amounts which would have to be paid in acquiring the right of way, supposing the railroad to be removed, are wholly beyond reach of any process of rational determination. The cost of reproduction method is of service in ascertaining the present value of the plant, when it is reasonably applied and when the cost of reproducing the property may be ascertained with a proper degree of certainty. But it does not justify the acceptance of results which depend upon mere conjecture. It is fundamental that the judicial power to declare legislative action invalid upon constitutional grounds is to be exercised only in clear cases. The constitutional invalidity must be manifest, and if it rests upon disputed questions of fact the invalidating facts must be proved. And this is true of asserted value as of other facts.

6. "The evidence in these cases demonstrates that the appraisement of the St. Paul and Minneapolis properties which were accepted by the master were in substance appraisals of what was considered to be the peculiar value of the railroad right of way. Efforts to express the results in the terms of a theory of cost of reproduction fail, as naturally they must, to alter or obscure the essential character of the work undertaken and performed. Presented with an impossible hypothesis, and endeavoring to conform to it, the appraisers—men of ability and experience—were manifestly seeking to give their best judgment as to what the railroad right of way was worth. And doubtless it was believed that it might cost even more to acquire the property, if one attempted to buy into the cities as they now exist and all difficulties that might be imagined as incident to such a 'reproduction' were considered. The railroad right of way was conceived to be a property *sui generis*, 'a large body of land in a continuous ownership,' representing one of the 'highest uses' of property and

possessing an exceptional value. The estimates before us, as approved by the master, with his increase of 25 per cent. in the case of the Duluth property, must be taken to be estimates of the 'railway value' of the land; and whether or not this is conceived of as paid to other owners upon a hypothetical reacquisition of the property is not controlling when we come to the substantial question to be decided.

7. "That question is whether, in determining the fair present value of the property of the railroad company as a basis of its charges to the public, it is entitled to a valuation of its right of way not only in excess of the amount invested in it, but also in excess of the market value of contiguous and similarly situated property. For the purpose of making rates, is its land devoted to the public use to be treated (irrespective of improvements) not only as increasing in value by reason of the activities and general prosperity of the community, but as constantly outstripping in this increase, all neighboring lands of like character devoted to other uses? If rates laid by competent authority, State or national, are otherwise just and reasonable, are they to be held to be unconstitutional and void because they do not permit a return upon an increment so calculated?

8. "It is clear that in ascertaining the present value we are not limited to the consideration of the amount of the actual investment. If that has been reckless or improvident, losses may be sustained which the community does not underwrite. As the company may not be protected in its actual investment, if the value of its property be plainly less, so the making of a just return for the use of the property involves the recognition of its fair value if it be more than its cost. The property is held in private ownership and it is that property, and not the original cost of it, of which the owner may not be deprived without due process of law. But still it is property employed in a public calling, subject to governmental regulation, and while under the guise of such regulation it may not be confiscated; it is equally true that there is attached to its use the condition that charges to the public shall not be unreasonable. And where the inquiry is as to the fair value of the property, in order to determine the reasonableness of the return allowed by the rate-making power, it is not admissible to attribute to the property owned by the carriers a speculative increment of value, over the amount invested in it and beyond the value of similar property owned by others solely by reason of the fact that it is used in the public service. That would be to disregard the essential conditions of the public use and to make the public use destructive of the public right.

9. "The increase sought for 'railway value' in these cases is an increment over all outlays of the carrier and over the values of similar land in the vicinity. It is an increment which cannot be referred to any known criterion, but must rest on a mere expression of judgment which finds no proper test or standard in the transactions of the business world. It is an increment which in the last analysis must rest on an estimate of the value of the railroad use as compared with other business uses; it involves an appreciation of the returns from rates (when rates themselves are in dispute) and a sweeping generalization embracing substantially all the activities of the community. For an allowance of this character there is no warrant.

10. "Assuming that the company is entitled to a reasonable share in the general prosperity of the communities which it serves, and thus to attribute to its property an increase in value, still the increase so allowed, apart from any improvements it may make, cannot properly extend beyond the fair average of the normal market value of land in the vicinity having a similar character. Otherwise we enter the realm of mere conjecture. We therefore hold that it was error to base the estimates of value of the right of way, yards, and terminals upon the so-called railway value of the property. The company would certainly have no ground of complaint if it were allowed a value for these lands equal to the fair average market value of similar land in the vicinity, without additions by the use of multipliers, or otherwise, to cover hypothetical outlays. The allowances made below for a conjectural cost of acquisition and consequential damages must be disapproved; and in this view we also think it was error to add to the amount taken as the present value of the lands the further sums calculated on that value, which were embraced in the items of 'engineering, superintendence, legal expense', 'contingencies', and 'interest during construction.'"

Conclusions to be Drawn from the Minnesota Rate Case.—Careful study of this case seems to indicate that the Court did not intend to exclude from railway lands any of those elements of cost which necessarily must be met in their acquisition; on the contrary, it is distinctly stated that

"It is impossible to assume that the company would be required to pay more than its fair market value. It is equipped with the power of eminent domain. * * * It would be free to stand upon its legal rights and it would not be supposed they would be disregarded."

Thus, the Court makes the cost of acquiring by condemnation the test. The utility cannot claim any more in the reproduction estimate than would be paid were all the land to be condemned, giving the owners its fair market value for all available uses and purposes.

Another point that is strongly brought out is the fact that land valuation must be capable of proof by reference to known and established criteria and must not be based on speculation or hypothesis. Among the paragraphs quoted by the Court from Mr. Cooper's testimony, is the following:

"'Q. * * * There is nothing in any of your exhibits which will show, nor are you now prepared to state, the difference in what might be termed the normal, true, ordinary market value of the lands to the ordinary individual, and the sum which you have fixed as the market value to the railroad company if it were now compelled to purchase? A. That is correct.'"

Again, after discussing all the elements of value which would be allowable in case of condemnation, the Court says:

"There is no evidence before us from which the amount which would be properly allowable in such condemnation proceedings can be ascertained."

and, in the next to the last paragraph quoted:

"The amount of the fair value of the company's land cannot be satisfactorily determined from the evidence."

This decision must be looked on as sounding a strong note of warning against adding elements of value which are hypothetical, or increments which are based on generalization or speculation.

The adoption of a sweeping rule and its application, when "it cannot be referred to any known criterion", and has "no proper test or standard in the transactions of the business world", is not endorsed in this case. The problem in the valuation of lands of the class under consideration is to find such a rule as will apply, as will be capable of direct proof, and as will be supported by the standard of ordinary transactions in the business world and by actual experience in the buying of lands for similar purposes under similar conditions.

Although the decision does not seem to be capable of such an interpretation as would exclude any elements of value which would actually be paid for in acquiring the land, it does unequivocally bar from consideration any intangible values which would attach to the property by reason of its new ownership and new use. No hypothetical "railway value" is to be considered. The decision, throughout the discussion of land values, is full of references to these speculative elements of value and the impossibility of their determination by any known rules. It would appear to be clear that the Courts will not approve any attempt to attach to land values, in connection with the valuation of physical property, "a speculative increment of value, over the amount invested in it and beyond the value of similar property owned by others solely by reason of the fact that it is used in public service."

There is one element of loss to the owner which is not mentioned in the decision, but the Committee feels sure that the omission was not intended to deny the propriety of including the element, because, as seems to be indicated in Paragraphs 6 to 10, the Court satisfied itself with the decision that an attempt had been made to show "railway value", "what the taker had gained", for the land, that this was improper, and that there was no evidence submitted to show what the real value might be, and hence that other questions which might be pertinent were not considered. The element to which reference is made is that of damage to the remaining property by reason of the taking of a part of it—severance damages.

There is a long line of Court decisions in condemnation cases, and some State statutes which sustain the inclusion of this element of

cost wherever condemnation is necessary. It is not believed by the Committee that this decision is to be construed as arbitrarily excluding this element of just compensation, or value to the seller, where it would occur, and would be capable of proper proof.

Although the decision seems to exclude another item of cost to the company—namely, the costs of acquisition—the Committee believes that this exclusion is apparent rather than real, and that a proper estimate of the costs of acquisition, which should be made on a rational basis, the result of experience in or knowledge of similar purchases, would undoubtedly have been approved. The testimony before the Court seemed to include all the elements of cost over "fair value" as "fair value", and then to add to this and resulting sums successive percentages for engineering, etc., contingencies, and interest; all of which appealed to the Court as creating a purely hypothetical or conjectural value.

Therefore, in spite of the quite positive character of the latter half of Paragraph 10, the Committee believes that this wording, taken in connection with the last three sentences of Paragraph 3, the second sentence of Paragraph 4, the well-known interpretation at law of the term "just compensation", and the character of the testimony offered in the Minnesota rate case, does not exclude properly estimated "fair market value", "severance damage" where it would be incurred, or proper "costs of acquisition", from an estimate of the cost of reproduction, however the Court may regard any of these items when finally determining "fair value".

All three are certainly elements of the cost recognized by all persons who have had to do with the purchase of properties for public utilities whether privately or publicly owned. The Court contented itself with showing that the evidence before it was to sustain the value as what the taker would gain, not what the seller would lose; and that this is not a proper basis for cost of reproduction.

The Meaning of the Term "Market Value."—The words "market value" in the ordinary acceptance of the term mean the price of a commodity in the open market, the price at which it is bought and sold.

The market value of land for general purposes, as used in condemnation and similar cases, has been defined many times. As comprehensive and satisfactory a definition as any is that of the Supreme Court of Kansas, as follows (Kansas City, etc., Rd. Co. *vs.* Fisher, 49 Kans., 17):

"The market value means the fair value of the property as between one who wants to purchase and one who wants to sell, not what could be obtained for it under peculiar circumstances when a greater than its fair price could be obtained, nor its speculative value; not a value obtained from the necessities of another; nor on the other hand is it to be limited to that price which the property would bring when forced

off at auction under the hammer. It is what it would bring at a fair public sale when one party wanted to sell and the other to buy."

Mr. Justice Hughes fixed condemnation as the measure.

"It is impossible to assume * * * that the company would be compelled to pay more than its fair market value. It is equipped with the governmental power of eminent domain. * * * The owner of each parcel would be entitled to receive on its condemnation its fair market value for all its available uses and purposes."

What is the measure of damages in condemnation? What would the seller of the land be compelled to take as the fair market value? Lewis in "Eminent Domain", Section 463, states:

"Measure of damages when entire tract is taken. This case presents but little difficulty, and, so far as we have observed, there is no difference in the authorities as to the proper measure of damages. A fair equivalent for any entire piece of property is its value in money."

Also, in Section 464:

"When part is taken, Just Compensation includes damages to the remainder. Upon this point there is entire unanimity of opinion. The constitutional provision cannot be carried out, in its letter and spirit, by anything short of a just compensation for all the direct damages to the owner of the lot, confined to that lot, by the taking of his land. The paramount law intends that such owner, so far as that lot is in question, shall be put in as good a condition, pecuniarily, by a just compensation, as he would have been in if that lot of land had remained entire, as his own property. How much less is that lot, and its erections thereon remaining, worth to the owner, as property to be used or leased or sold the day after the part was taken, to be used for the purposes designed, than the whole lot intact was the day before such taking (Bangor & Piscataqua R. R. Co. *vs.* McComb, 60 M. E., 290). In considering damages to the remainder however, the whole remainder must be taken into account. If part is damaged and part benefited, the question will be whether the whole is worth less than before the taking."

The same use of the word "just" is to be found in nearly every State Constitution wherein "just compensation" is used to express the measure of damages for taking of property under the right of eminent domain.

The value to be used, therefore, is the fair market value of the property taken and the damages to that not taken, and "just compensation" shall take into account all elements of value heretofore discussed.

The Committee is of the opinion that in estimating cost it is not error to consider each parcel originally acquired, and to attach to it such reproduction cost as would be fixed in condemnation, including severance damages, where severance would actually occur, and that the reproduction cost so found should be accepted as significant. To

assume arbitrarily a rule of valuation, based on the use of hypothetical multiples, and apply it to the entire acreage of lands in a State cannot be sustained. The failure to point out what proportion of lands bear severance damages, and the failure to support by actual recent costs or actual recent experience in the acquisition of similar lands, and the further failure to indicate the relation existing between the price per acre measure when applied to utility lands, many if not most of which have a large element of consequential damages, and the price per acre measure applied to entire tracts taken or tracts sold for general purposes, is likely to be serious and defeat the object of the valuation.

Value to Owner and Taker.—The Element That Must be Excluded in Valuation.—Turning to Boston Chamber of Commerce *v.* Boston (217 U. S., 189, 195), we find the origin of the expression of the quoted paragraph, "as to what the owner had lost, and not what the taker had gained", and the facts in that case clearly indicate that in condemnation cases the inquiry must be, "What has the owner lost?"

It is somewhat difficult to understand that the taker may fairly obtain something with which the owner does not part, but, taken in connection with Paragraphs 478 and 479 of Lewis on "Eminent Domain", and with decisions heretofore quoted, it seems to be quite apparent that Mr. Justice Hughes made what he meant sufficiently clear when he said:

"The owner would not be entitled to demand payment of the amount which the property might be deemed worth to the company, or of an enhanced value by virtue of the purpose for which it was taken, or of an increase over its fair market value, by reason of any added value supposed to result from its combination with tracts acquired from others, so as to make it a part of a continuous railroad right of way held in one ownership."

This language harmonizes with the well-settled principles set forth by Lewis, and shows that there may be something gained by the taker that is not properly a part of the damages which the owner sustains. This additional value is not properly to be included in the reproduction estimate.

The subject is gone into at some length in U. S. *v.* Chandler-Dunbar (229 U. S., pages 79-81), where a strategic value was claimed for certain property. Therein, the Court said:

"That the property may have to the public a greater value than the fair market value, affords no just criterion for estimating what the owner should receive. It is not proper to attribute to it any part of the value which might result from a consideration of its value as a necessary part of a comprehensive system of river improvement which should include the river and the upland upon the shore adjacent. * * * A 'strategic value' might be realized by a price fixed by the necessities of one person buying from another, free to sell or refuse as the price

suited. But in a condemnation proceeding, the value of the property to the Government for its particular use is not a criterion. The owner must be compensated for what is taken from him, but that is done when he is paid its fair market value for all available uses and purposes, Lewis, Eminent Domain, 2d Ed., Sec. 706; Moulton *v.* Newburyport Water Co., 137 Mass., 163, 167; United States *v.* Senfert Bros. Co., 78 Fed. Rep., 520; Alloway *v.* Nashville, 88 Tenn., 510, 514; United States *v.* Honolulu Co., 122 Fed. Rep., 581."

The Committee interprets the decision of the Court to support quite clearly the contention that all elements of value and damages to the seller considered in condemnation must be considered in valuation, but that special values due to increased earning power by reason of a new use are not approved in these decisions. This feature is especially emphasized in City of New York *vs.* William Sage, Jr., (229 U. S., 372).

In this case, the Commissioners appointed by the City reported that:

"The sum of $7 624.45 for lands and buildings and the further sum of $4 324.45 for reservoir availability and adaptability being a grand total of the sum of $11 948.90 is the sum ascertained and determined by us * * * to be paid to the owners of and all persons interested in said land for the taking of the fee thereof, designated * * * as Parcel 733."

The report was confirmed by the Circuit Judge (190 Fed., 413) and by the Circuit Court of Appeals (124 C. C. A., 251, 206 Fed., 369) but was reversed by the U. S. Supreme Court, which said (229 U. S., 372):

"Upon an inspection of the record it appears to us, as the language of the Commissioners on its face suggests, that their report does not mean that the claimant's land had a market value of $11 948.90—that it would have brought that sum at a fair sale—but that they considered the value of the reservoir (site) as a whole and allowed what they thought a fair proportion of the increase, over and above the market value of the lot, to the owner of the land, subject to the opinion of the Court upon the point of law thus raised. * * * But the only explanation of the separation of items by the Commissioners is that they were not prepared to say that the market value of the lot was $11 948.90, seeing that the claimant bought it a few days before for $4 500, but that they thought the additional value gained by the City's act should be taken into account and shared between the City and the owner of the land—a proposition to which we cannot assent (Minnesota Rate Cases, 230 U. S., 352, 451; McGovern *v.* New York, 229 U. S., 363, 372)."

* * * * * * *

"The decisions appear to us to have made the principles plain. No doubt when this class of questions first arose it was said in a general way that adaptability to the purposes for which the land could be used most profitably was to be considered; and that is true. * * * But it is to be considered only so far as the public would have considered

it if the land had been offered for sale in the absence of the City's exercise of the power of eminent domain.

"The fact that the most profitable use could be made only in connection with other land is not conclusive against its being taken into account, if the union of the properties necessary is so practicable that the possibility would affect the market price. * * * But what the owner is entitled to is the value of the property taken, and that means what it fairly may be believed that a purchaser in fair market conditions would have given for it in fact—not what a tribunal at a later date may think a purchaser would have been wise to give, nor a proportion of the advance due to its union with other lots.

"The city is not to be made to pay for any part of what it has added to the land by thus uniting it with other lots if that union would not have been practicable or have been attempted except by the intervention of eminent domain.

"Any rise in value before the taking, not caused by the expectation of that event, is to be allowed, but we repeat, it must be a rise in what a purchaser might be expected to give."

* * * * * * *

"The enhanced value of the land as a part of the Ashokan Reservoir depends on the whole land necessary being devoted to that use. * * * If the parcels were not brought together by a taking under eminent domain, the chance of their being united by agreement or purchase in such a way as to be available well might be regarded as too remote and speculative to have any legitimate effect upon the valuation."

This case well illustrates the fact that in condemnation proceedings the owner is not entitled to the enhanced value of the land which comes to it as a part of the enterprise for which it is taken; that is to say, the additional value gained by the company's act of taking the land and combining it with other parcels should not be taken into account, and the owner is not entitled to participate in such additional value so gained. The company is not bound to pay any part of what it adds to the land by uniting it with other tracts. In determining the reproduction cost—what the owners are entitled to receive—adaptability to the purposes for which the land can be used most profitably may be considered, but only so far as the public would have considered such adaptability if the land had been offered for sale in the absence of the company's exercise of the power of eminent domain. The available uses and purposes referred to may not properly include any more than what it may be fairly believed a purchaser under fair market conditions would pay for the land in advance of its union with other lots for more profitable use. Any value which exists before the taking of the land, and which is not caused by the expectation of that event, is to be allowed.

Normal Cost of Public Utility Lands as Compared with General Land Values.—As has already been pointed out, sales of land for

general purposes are made under conditions which are very different from those that prevail in the case of sales for railroad or reservoir lands. In one case, the sale is of the entire parcel, both parties to the transaction being willing parties, the buyer being free to take or leave the bargain. In the other case, the purchaser must take this particular parcel or part thereof and no other. He may condemn, but if he takes only a part, he must include the element of severance and other damages in his price. These considerations lead the Committee to the conclusion that fair comparison can only be made with similar purchases, that is, railroad land costs for inclusion in a valuation by the reproduction method must be determined by a study of railroad purchases of lands of the same class, in the same locality. It may be that in certain districts the cost of lands for railroad right of way exceeds little, or not at all, the cost of lands for farming or other general purposes, while a few miles away the development of the country is such that very heavy severance damages will be incurred, thereby increasing the resulting cost per acre to several times that of the general purpose land. It devolves on the evaluator, therefore, to support his estimate by actual history as to the extent of inclusion of damages, and by actual recent costs of like accumulations of property in the same or contiguous territory or in territory where all general conditions are similar.

That, as a general rule, a right of way for a railroad or a body of lands acquired for a dam or reservoir does cost more per acre or square foot than other lands is clearly shown by a study of records of actual costs of such tracts.

Professor David Friday, in making the land valuation of the Pere Marquette Railroad, in Michigan, cites several instances of land purchases, as follows:

(1).—The right of way from Greenville to Stanton, purchased in 1900, cost $22 435. This includes several $1 considerations and does not include such costs of acquisition as services of real estate agents, registration fees, legal and recording fees, and other similar items of expense. The value of this record, on the basis of contiguous lands, would be only $6 738, manifestly an absurdly low figure in view of the fact that land in this district has increased in value more than 60% since the date of purchase.

(2).—The right of way from Belding to Lowell, purchased at various times between 1892-1900, cost $32 518. The present value, on the basis of adjoining lands, is $13 136. Here, also, the increase in value has been large.

(3).—The right of way from Oak to Delray, purchased in 1891-1892, cost $191 013. The present value, on the basis of adjoining lands, would be $121 120. The proximity of this land to the City of Detroit has increased general values by more than 100% since the acquisition of the right of way.

(4).—The land for the right of way for the new track at New Buffalo, consisting of 199 acres, was purchased in 1911 at an average cost of more than $185 per acre. This is at least three times the unit value of adjoining lands.

The following, also cited by Professor Friday, serve as further examples of the cost of railroad rights of way in various locations and under various conditions:

The Benton Harbor-St. Joe Interurban Road, from Benton Harbor to Coloma, Mich., a distance of 12 miles, bought, in 1910, 100.46 acres for its right of way at an average cost per acre of $267.30. This was bought through a territory which desired the road and in which land was worth from $60 to $100 per acre.

The Milwaukee, Sparta and Northwestern Railway in the State of Wisconsin purchased the 5 127 acres in its right of way and station grounds for $1 129 531. The market value of these lands on the basis of values of adjoining lands was $361 015, or less than one-third the amount actually paid therefor.

The Chicago, Burlington and Quincy Railway bought, some years ago, land in the City of Moline for $112 000. The market value of these lands, as estimated on the basis of adjoining values, was $66 600.

The "Soo" line paid for 3 460 acres in Wisconsin, purchased in extending its road from Glenwood north to the State line and from Thief River Falls west to the State line, the sum of $190 950. The value per acre of land in the immediate vicinity as estimated by the Wisconsin Tax Commission was $19.30; the price paid by the Railroad Company was $55.20, or 2.85 times the ordinary value.

Purchases for the Illinois Central through Mower and Freeborn Counties in Minnesota some years ago, amounting to 228 acres, cost the Railroad $125 per acre, while the adjoining lands were estimated by the Minnesota Railroad and Warehouse Commission as having a value of $49 per acre.

The Hastings and Northwestern Railroad Company, a subsidiary of the Union Pacific Railroad, purchased its right of way between August, 1912, and May, 1914, at a cost of $221 355.06. The value of this land, on the basis of unit values of adjoining land was $96 729.

In 1903, an extended series of investigations of actual considerations paid by railroads for rights of way for new lines, changes of line, and for lands for widening right of way, were made by Mr. Van Ranst Pond and H. E. Riggs, M. Am. Soc. C. E.

These investigations were supported by a large mass of testimony given by registers of deeds, land owners, and others, in the Michigan tax cases. There are very few condemnations in the many hundreds of transfers investigated. Some typical examples of these data follow:

The Jackson and Battle Creek Railway was an electric interurban line, partly paralleling the highways, and for a number of miles being parallel with and adjoining the Michigan Central right of way—all country lands; average considerations to owners in Jackson County, $239.53 per acre; in Calhoun County, $218.74 per acre. Farm land values in these counties range from $60 to $100 per acre, with $75 to $80 per acre as a liberal average.

In Monroe County, Michigan, several lines of road have been built since 1900. This is good farming land worth from $80 to $100 per acre. Considerations (average for the entire County) were paid for railroad lands as follows: Flint and Pere Marquette, $215.21; Toledo and Monroe (electric), $461.13; and the Detroit and Toledo Shore Line, $262.49.

The Grand Trunk Railway made certain changes of its lines near Flint and near Battle Creek, which involved in each instance building several miles of new line. This was far enough away from the old line to get out of the same farms and, except at the extreme ends, there were no complications due to crossing lands already cut. There were no condemnations. The line near Flint, in Genesee County, was entirely across good farm land worth from $100 to $115 per acre. The average paid per acre was $337.56. West of Battle Creek, in Calhoun County, the land was sandy and not high-class, and worth from $60 to $85 per acre. The average consideration was $491.13 per acre. In neither case were values affected by proximity to the cities, as there was no plotting of new additions near the city nor of either cut-off.

In 1896, the Ann Arbor Railroad built a new main line 10 miles north in Washtenaw County. This line left the old line about 3 miles north of Ann Arbor and was several miles west of the old road. It was wholly in farming lands, part of poor quality. Values ranged from $50 to $90 per acre, averaging about $75. The considerations averaged $285 per acre.

The foregoing examples are only a few of the many investigated in Michigan. In these, the railroad lands, no matter how acquired, cost

from two and one-half to six times as much as general lands, due, doubtless, in great measure, to the amount of severance damages paid by the companies, but which elements of cost were not segregated in the work in Michigan.

In none of the examples cited are costs of acquisition or overhead cost included.

Among the illustrations presented by the railroads to the Interstate Commerce Commission in 1915 was a statement showing, for a number of roads in several States, the actual cost of recent acquisitions of railway lands. This statement includes purchases on a total of more than 465 miles of line, and is summarized as follows:

Miles of line.	Total purchase, exclusive of costs of acquisition.	Bare land value, based on general values.	Percentage of land value to total cost.	Other elements of cost.	Percentage of other elements to total cost.
Right of way, 465.4.	$7 868 213	$3 857 783	49	$4 009 831	51
Yards and terminals.	2 005 158	1 144 774	57.2	856 384	42.8

These figures indicate so clearly that railroad purchases exceed purchases for general purposes that it would appear, with a proper presentation as to the percentages of the total acreage affected by other elements of cost, and proper support by reference to actual recent railroad costs under conditions similar to those under investigation, that the proof might be considered to be convincing as to the relation of actual costs to general values of lands.

That lands belonging to other classes of utilities present the same relation to general lands is more difficult of exact illustration on account of the scarcity of reliable records.

Certain records are available, showing the considerations paid for lands and the amounts paid for damages and incidental costs other than land in connection with the acquisition of lands.

New York City—Catskill Water Supply—Cost of Lands.*

Direct cost of 21 327 acres	$10 699 946;	average	$509	per acre
Interest (actual)	1 648 801	"	78	" "
Indirect and consequential damages.	294 801	"	14	" "
Fees, legal, and sundry expense.	4 188 795	"	200	" "
Total cost of lands	$16 808 344;	average	$801	per acre
Excluding interest			$723	" "

The cost of acquisition, or overheads, excluding interest, aggregates 39.26 per cent.

* Based on Board of Water Supply figures of December 31st, 1914, litigation still pending as to value of 345 acres.

An analysis of this figure is of great interest as showing the same wide range between cost of land and expenses that occur in railroad lands.

Division of work.	Acres.	Cost of land per acre, award.	Expenses per acre.	Expenses, percentage of land cost.	Total per acre cost.
Ashokan Reservoir	15 221	$ 243	$ 155	61.41	$ 398
Northern Aqueduct	1 637	470	428	91.08	898
Kensico Reservoir	3 182	742	404	53.17	1 146
Hill View "	163	8 796	3 871	44.01	12 667
Southern Aqueduct	957	1 987	787	39.66	2 774
City Aqueduct, 90 acres owned by city (bought)	76	6 585	962	15.03	7 547

A recent case in Michigan is very interesting, as throwing light on the cost and value of lands for flowage purposes.

A power company was organized to take over a tract of 17 000 acres of land on the Sturgeon River and build two hydro-electric plants. In 1911, the Duluth, South Shore and Atlantic Railway valuation placed the company's right of way in these counties (Houghton and Baraga) at from $25 to $45 per acre for country lands near the proposed site of the reservoir (about 2 to 5 miles distant). The State's attorney contended for values of from $5 to $10 per acre. Actual land sales throughout that part of the country occupied by reservoir and railroad had ranged from $2 to $10 per acre plus the value of the timber on the lands, or an average, land and timber, of about $12. Under the law, the power company applied to the Michigan Railroad Commission for authority to issue stock and bonds. The opinion, by Commissioner Lawton T. Hemans, is dated June 11th, 1913, and is in part as follows:*

"We believe that the lands described in the application herein should be capitalized at their fair market value, as represented by their initial cost, plus reasonable compensation for the time, energy, and ability bestowed in their acquisition, for the risk involved, and for the use of the capital invested during the interval that must elapse between the inception of the project, in carrying forward negotiations and purchases of distinct parcels, and their final utilization as an assembled whole. It may be contended that the various elements which it is suggested should form the basis of the capitalization allowed for lands and flowage rights are, except as to initial cost, as indefinite as capitalization computed upon any of the bases which the Commission rejects, but the Commission is persuaded that with the various elements stated, disproportion between them, if any exists, becomes more apparent with correspondingly greater certainty in arriving at correct values.

* Northern Michigan Power Company, Orders and Opinions, Michigan Railroad Commission, Vol. 2, No. 1, p. 25.

"It is not to be understood from the foregoing that the Commission holds that the initial cash payment for lands of the character involved is to be considered, of necessity, the all controlling factor in the fixing of their value for purposes of capitalization. Cases can be readily imagined where the cash cost of the physical property embraced in the lands and flowage rights was the least important of the factors enumerated. Indeed, the tract of land under consideration partakes strongly of this character. It is located in a comparatively undeveloped portion of the State, although in close proximity to the copper mining district of the State and possessed of some timber value, the physical features of the greater portion of the tract are such as to make it unsuited for agricultural purposes, compared with like areas alike suitable for water power development in other and more populous districts, the market value of the land is markedly less. The record does not disclose the exact cost of the assembled lands, although it does appear that they have been acquired at prices ranging from ten to one hundred and seventy dollars per acre, and that more than thirty thousand dollars was expended for lands at the last named sum. We are satisfied, from the record, that the sum of five hundred thousand dollars would amply cover the cash outlay for the purchase of the lands. But it appears that the work of acquiring the titles has been in progress since 1902, a period of more than ten years. If interest upon the real estate investment be allowed at six per cent. for half the time, which would seem to be just, it represents the sum of one hundred and fifty thousand dollars. The extended area of the lands necessary for this development have required, as the record discloses, the making of several surveys, the prosecution of legal proceedings, and extended negotiations with more than one hundred separate owners of distinct parcels, who have been found as widely scattered as the boundaries of the Union. Under the laws of this State, the powers of eminent domain are not accorded corporations which seek to develop hydraulic utilities. The last description of land necessary for the purposes of the enterprise must be acquired by purchases before the project is assumed, and while the element of risk involved in the ultimate purchase of so large a body of lands, held under such diversity of ownership, is difficult to estimate in dollars and cents, it is still a risk that is very real, and which enters materially into the value of the property acquired."

The testimony in this case shows that the lands varied in cost from $10 to $170 per acre, averaging about $30. This was more than two and one-half times the average price per acre of lands sold in ordinary tracts. The costs of acquisition, allowing a moderate per diem for the actual time of the promoter plus the expense, were equal to the consideration paid to owners of land.

The examples cited are not exceptions. They are typical of all the cases which have come to the attention of the Committee. There is no general relation between the normal cost of lands acquired for railroad or reservoir purposes, and the market value of farm lands, or other general purpose lands, which can be considered fixed and definite.

Such a relation, however, may be established for limited districts. No rule can be formulated or multiple determined on, which can be universally applied, or even generally applied in any one State.

There have been, in the past, instances of the construction of railroads in timber tracts, or in the western prairie States, in advance of settlement, where the benefits to be derived from the line equalled or in cases far outweighed any damages; and, in such cases, it is easy to conceive that the price paid for the land would not greatly exceed the general average value of lands in the district.

Such conditions rarely exist in an old settled community. For many years the conditions prevailing in all such communities have made the normal cost of railroad and reservoir lands higher when the attempt is made to compute them on the basis of a price per acre.

Costs of Acquisition.—To What Extent Includable in Valuation?—The Interstate Commerce Commission in its classification of Investment in Road and Equipment, effective July 1st, 1914, includes as Items of Expense relating to land the following:

Abstracts
Appraisals
Arbitrators in condemnation cases
Commissions paid to others
Condemnation expenses including Court costs and special counsel fees
Damages to property of others
Deferred payments for right of way
Ditches for waterways when part of consideration
Judgments and decreed costs to clear or defend titles
Notarial fees
Plats
Premiums on condemnation bonds
Recording deeds
Payments for relinquishments of cattle passes and other rights
Removal and relocation of buildings and other structures not purchased
Rent of land when part of consideration for purchase
Right-of-way agents' compensation when engaged solely in acquiring right of way
Taxes accrued and assumed at the time of purchase.

Every one of these items is a necessary expense, not all incurred in connection with every parcel, but every parcel involving some of them, and all are chargeable to capital account as a legitimate cost.

The amount is considerable, in some cases a very large percentage of the purchase price of the parcel of land, but it is somewhat difficult to compute, for when it is reduced to a per acre basis or a percentage of cost basis it is extremely variable. The cost of condemning, or acquiring by purchase, a small fragment of land one-tenth of an acre in area may be as great as the cost of similarly acquiring a farm of many acres.

Examples of actual cost of acquisition of lands are of value as showing the extreme variations in percentage and per acre rates of such costs.

Land Purchases of the Buffalo, Rochester and Pittsburgh Railway.

Year ending June 30th.	No. of parcels.	Total acres.	Consideration for land.	Total costs of acquisition.	Cost of Acquisition: Percentage of consideration.	Per acre.	Per parcel.
1910.....	35	165.789	$20 086	$5 828	2.9	$35.30	$166
1911.....	16	58.694	16 604	1 521	9.1	26.00	95
1912.....	29	60.732	33 506	1 718	5.1	26.50	59
1913.....	63	109.705	73 469	3 486	4.7	31.70	55
1914.....	24	28.730	50 292	3 677	7.3	126.40	153
Total and average.	167	423.646	$193 958	$16 231	8.38	$38.31	$97

These variations in costs of acquisition in percentages from less than 5 to 29, in cost per acre from $26 to $126, in cost per parcel from $55 to $166, are such as to show the necessity of careful investigation of varying local conditions before fixing the estimated amounts.

Land purchases of the Chicago and Northwestern, on one of its new lines, show ratios as follows: 3 620.41 acres; land consideration, $594 169; expense, $72 685; being 7.62%, or $20 per acre.

Statistics presented by Mr. Thomas W. Hulme, in the argument recently held before the Interstate Commerce Commission, show purchases by twenty different corporations for forty-five different pieces of construction. The percentages vary from a minimum of 0.86 to a maximum of 38.4.

Total consideration, all lines, $8 389 730.

Costs of acquisition, $452 656, average 5.4 per cent.

Water-works properties afford an illustration of a class of properties carrying a high cost of acquisition. This is shown by the land costs of the New York City Catskill water supply.* The whole matter may be briefly summarized, as follows:

Amount of awards for land, including payments for land purchased.....	$10 669 946
Awards for indirect damages.....	135 007
Estimated awards for land and damages during 1915, proportionate part of $500 000	323 000
Total payments for land and damages.....	$11 127 953

* Information furnished and arranged by Allen Hazen, M. Am. Soc. C. E., from records and published reports and with the aid of engineers of the Department.

Brought forward		$11 127 953
Expenses of acquiring land not including interest, as per Auditor's report	$4 188 795	
Do. in connection with damages awards	159 794	
Do. in connection with estimated awards during 1915, proportionate part of $500 000	127 000	
Total expenses		$4 475 589
Sum, being the cost of land, with expenses of acquiring it, but not including interest		$15 603 542
Interest on awards allowed by Commissioners and included in the Auditor's report, as part of the cost of land	$1 648 802	
Estimated interest on awards allowed for 1915, proportionate part of $500 000	50 000	
Total interest on awards		$1 698 802
Total actual payments for property to January 1st, 1916, including an estimate of $500 000 for payments in 1915		$17 302 344
Cost of carrying, after acquiring, up to the time when the system is to be put in service, taken as January 1st, 1916:		
Taxes paid	$216 534	
Taxes estimated for 1915	25 000	
Interest on amounts paid to January 1st, 1916, at 4.15%	3 746 652	
Total carrying charges		$3 988 186
Total cost of land and damages, with carrying charges up to date of first use of property, January 1st, 1916		$21 300 530

Calculating the percentages corresponding to these figures, it appears that the expenses of condemnation amounted to 40.1% of the awards and costs; that the interest on the awards allowed by the Commissions and included by the Auditor as part of the cost of the

land amounted to 10.85% of the awards and expenses; that the taxes paid by the city amounted to 1.5% of the cost of the preceding items; and that the interest on the various payments, from the dates of the payments until the property is put into use on January 1st, 1916, will amount to 21.3 per cent.

As a result of all these charges, the cost of the land to the city, with carrying charges to the date of first use, is 91.3% greater than the purchase prices and awards. In other words, the overhead charges on the land are 91.3 per cent.

The expenses of acquiring land as stated by the Auditor, in the sum of $4 188 795, are given below.

This particular case represents the acquisition of land by a city under a law which proved in its execution to be unusually burdensome. It is of value for illustrative purposes, on account of the prominent character of the work and the detail as to costs.

The total costs aggregated 39.26% of the purchase price, and were:

		Per cent.
Advertising	$433 946.46	4.16
Commissioners' fees	1 062 498.72	9.96
Expenses of Commissioners	79 898.40	0.75
Stenographers and clerks	62 606.70	0.59
Special counsel fees	358 679.20	3.36
Special counsel expenses	27 493.06	0.26
Obtaining orders	8 375.00	0.08
Closing title	1 370.00	0.01
Counsel fees on appeal	3 275.00	0.03
Costs on appeal	4 499.97	0.04
Searching titles	211 138.64	1.98
Preparing abstracts	125 312.50	1.17
Appraisers' fees	482 512.53	4.52
Appraisers' expenses	5 792.72	0.06
Rent of New York office	26 269.40	0.25
Stenographers' services and printing testimony	265 684.24	2.49
Counsel fees of parcel owners	384 405.99	3.60
Expenses and disbursement of parcel owners	261 275.36	2.45
Engineering salaries and expenses	373 760.65	3.50
	$4 188 794.54	39.26

Another detailed statement regarding the acquisition of land by a State commission, under a law which was not burdensome, is afforded by the experience on the Metropolitan Water-Works of Massachusetts. In this case the principal acquisition of land was for the Wachusett

Reservoir, the construction of which was not started until several years after other portions of the work were begun. The policy was followed, therefore, of acquiring this land as it could be obtained by negotiation, without resorting to condemnation except in special cases. Much of the land was acquired in this way, considerably in advance of the needs of the work, at a much lower price than it would have been obtained by condemnation, but with an increase in the charges to taxes and interest during construction. From the accounts of all real estate purchased, from the initiation of the work in 1895 to December 1st, 1909, the following summary is obtained:

Total paid directly for real estate, including mill properties and water rights acquired by negotiation		$3 390 723
Awards for real estate condemned, including interest to the date of payment		170 446
Total direct payments for real estate		$3 561 169
Legal and expert services, including conveyancers, experts, appraisers, and court expenses, in acquiring above real estate, exclusive of counsel fees.	$179 944	
Estimated proportion of total administration expenses attributable to real estate	60 000	
Engineering expenses are shown by the accounts to amount to $83 582 for the Wachusett Reservoir alone. Assuming that other real estate acquisition would require the same percentage of engineering expenses, the total would be	104 700	
Estimated expenses for counsel in real estate cases, charged to the general expenses of the State, and, therefore, not included in the water-works accounts	20 000	
Estimated expenses for financing and financial officers, attributable to real estate and charged to the general expenses of the State and, therefore, not included in the water-works accounts	20 000	
Total cost of acquisition		$384 644

Brought forward	$384 644
Total cost of real estate, exclusive of taxes and interest during construction	3 945 813
Taxes during construction	68 182
Interest during construction on Wachusett Reservoir, real estate alone, on the basis of 5% compounded annually, would have amounted to $957 000, or 31% of the expenditures for this real estate, including incidental expenses, to the time when the reservoir was first operated. It is estimated that the interest during construction on other real estate would have increased this sum to	1 000 000
Total cost of real estate	$5 013 995

Calculating the percentages corresponding to the different figures, it appears that the incidental expenses of acquiring the real estate amounted to 10.8% of the direct payments for it; that the taxes paid during the construction of the works amounted to 1.9% of the direct payments; and that the interest on the various payments, from the dates of the payments until the property was put in use, on a 5% basis, would have amounted to 28.1% of the direct payments.

As a result of all these charges, the cost of the real estate, with overhead and carrying charges to the date of the operation of the property, is 40.8% greater than the direct payments for it.

The incidental expenses of 10.8% previously given may be subdivided as follows:

	Amount.	Percentage.
Legal and expert expenses, including counsel fees	$199 944	5.62
Administration	60 000	1.68
Engineering	104 700	2.94
Financial expenses	20 000	0.56
Totals	$384 644	10.80

The records of the Metropolitan Water-Works show the relation between the value of the real estate of the Wachusett Reservoir, exclusive of mill property, for general purposes, and the sums which were paid for it.

Before deciding on the project, an estimate of cost of the required real estate was made. To obtain a basis for this estimate, local experts were secured in each town, who went over all the lands, parcel by parcel, appraising the value of land and buildings separately. Their instructions were to value the property very liberally, and the results

obtained were much above its value as determined by sales which had been made about that time. Some of the experts owned property which would be taken for the reservoir. The assessed valuations in this district are supposed to be well up toward the full value of the property, but the value fixed by the experts was one and three-quarter times the assessed value of the property. As a result of the appraisal by the experts, the aggregate valuation obtained was, as follows:

Land, 4 772 acres.....	$244 000	
Buildings	453 000	$697 000

The cost of acquiring the same property, exclusive of overhead charges, was $1 182 000, equal to $485 000, or 69% in excess of the original liberal appraisal of the value.

It is to be noted that this is a case where the appraised general value of the buildings was nearly twice that of the land. The accounts do not show a separation between the amounts paid for buildings and land, but if these amounts could be obtained, they would show that the sums paid on account of the land, in excess of the appraised value, would be a large part of the $485 000 excess, for the reason that it is difficult to sustain a claim for the value of a building much in excess of its value for general purposes.

The 69% previously stated is, therefore, probably much more than the excess cost of the buildings, and much less than the excess cost of the land, over their values for general purposes. It is to be noted, also, that in the acquisition of this property most of the land was taken in whole tracts or parcels, so that there were no severance damages in connection with such portions of the property.

The accounts do not show the sums paid for strips of land taken for aqueducts or pipe lines in comparison with general land values, but the original estimate of the cost of these strips was obtained by multiplying the general value of the land by six, and adding a percentage for overhead charges. The actual cost corresponded very nearly to the estimate.

An illustration which is in marked contrast to the New York Catskill water supply is that of a railroad line in friendly territory, and is as follows:

LAND PURCHASES.

Details of Knife River Branch, N. Dak., Northern Pacific Railway.

	Mercer County, N. Dak.	Dunn County, N. Dak.
Valuation of naked land per acre, made by real estate men and bankers, not by employees.....	$22.07	$22.13
Total	$15 709.42	$12 065.12

	Mercer County, N. Dak.	Dunn County, N. Dak.
Total number of deeds	86	63
Total acres conveyed	784.97	615.55
Average acres per transfer	9.12	9.77
Total consideration	$24 460.	$17 106
Average consideration per acre	$35.00	27.80
Total number of considerations	4	None
Acres condemned	70.83	....
Award in condemnation cases	$1 966.52	....
Average per acre, award	$27.70	....

Summary, both Counties:

Total naked land value	$27 774.54	62.32%
All other elements of value	16 791.89	37.68%
Total purchase price	$44 566.43	100%
Expense of acquisition	$9 258.79	
Percentage expense of acquisition to purchase price	20.77	

Expense of Acquisition:

		Average per deed.	Average per acre.
Services and expenses, right of way agents	$6 790.75	$46.19	$4.85
Services and expenses of attorneys.	705.69	4.80	0.50
Costs of abstracts and opinions of title	1 227.80	8.35	0.93
Cost of recording deeds	329.55	2.24	0.23
Automobile hire	205.00	1.40	0.15
	$9 258.79	$62.98	$6.66

This case affords an illustration of the extremely favorable cost of acquisition. The line was desired by the people. There was but little contest, as two of the condemnation cases were to acquire land owned by the State of North Dakota, the other two being against land companies. Every individual owner was settled with, yet, in spite of these facts, the land cost was $27 774.54, while "other elements" plus cost of acquiring was $26 050.68, or 93.8% of the bare land price.

The conclusion to be drawn from a study of all the figures available to the Committee is that costs of acquisition, like the element of damages in the consideration, are variable, that each valuation must stand by itself, and that, in fixing a figure for land reproduction, the appraiser must take into account all special local conditions. It is perfectly clear that no general average percentage which is universally applicable is to be derived from these figures.

Conclusions.—Cost of Reproducing Land Holdings.—Separate parcels of land, such as one or two lots, or a tract of not unusual size not connected with other lands by the utility in a strip or body, and where freedom of choice in selection may be exercised, should be valued on the same basis as other lands in the vicinity used for general purposes, plus a reasonable allowance for costs of acquisition and other proper overhead costs. The normal market value of parcels of like size, character, and availability, in the immediate vicinity should be accepted as a basis for reproduction cost determination.

Where large tracts have been acquired, made up of a number of separate entire parcels, the history of the transaction should be completely investigated and allowance made in the estimate for costs of acquisition and costs of vacation of streets or acquisition of other rights incident to the use of the property as a whole. Such further allowances should be made, as may seem to be justified by experience, to cover excess costs over and beyond the normal market values which existed at the beginning of the project and were due to fluctuations of price or other causes beyond control. Normal market values of similar and near-by property as of the time of appraisal should be the basis, with such additions as are warranted by the investigation in each case.

In the consideration of values of lands, such as rights of way for a railroad, an electric railroad, an aqueduct, or other like property, or, such as lands for a water-works or an hydro-electric reservoir, freedom of choice is restricted, and where all the land must be acquired, radically different methods must prevail.

In the valuation of lands of this class, a clear distinction should be made between:

(*a*) Lands where the entire tract or parcel is taken and there is no element of severance damages.
(*b*) Lands where only a portion of the tract is taken and where the element of severance damages is present,

and the extent of lands of both classes should be shown.

Historic conditions, where ascertainable, should be given due weight in fixing the extent and character of the severance. The estimate of reproduction should include a proper figure for such as was actually incurred, but should not include allowance to cover speculative or hypothetical damages on account of changed conditions. All elements of value to the seller, including recognition of all damages to the portions of his property not taken, which would prevail in the case of condemnation of the lands under reproduction, should be considered.

A determination of the relation existing between actual recent acquisition costs and the normal market value of the lands out of

which the strip or parcel was taken at the time of acquisition is undoubtedly possible in many cases. It would seem that where such relation can be determined, its use in connection with the normal market values of these or similar lands at the time of appraisal should establish a reasonable cost of reproduction.

There must be kept in mind the fact that no general rule can be established capable of universal application. Conditions that will apply in the acquisition of railroad lands in a large city, or in a fully developed rural community where lands are subdivided into relatively small tracts and with many improvements upon them, may not be at all applicable in districts where lands are held in large tracts with few improvements, such as grazing districts or a heavily timbered district.

The determination of the figure to be used must be based on full consideration of present normal values of general purpose lands, recent purchases by the same or other companies of similar lands in the vicinity or in districts of like characteristics, the extent to which damages enter into the cost of lands, and the amount and character of costs of acquisition and overhead charges. All elements of costs—normal market values of individual parcels, including improvements thereon, severance and consequential damages, enhanced prices induced by active demand, expenses of acquisition and overhead costs, and any other real items of cost which would be included in case of purchase—should be included in reproduction, but no allowance for special values coming after the acquisition of the property, on account of its new use or on account of a greater earning power under the new use, or any other hypothetical "value", should be included.

Estimates should be based on prices and values as of the time of appraisal, be they higher or lower than those prevailing at the time of original acquisition. The thing sought in reproduction is the fair cost of acquiring the property as of the date of valuation.

The treatment of the valuation of land holdings is not yet thoroughly crystallized, therefore the valuing engineer will do well to confer with counsel upon the interpretation of past Court decisions and the legal principles which are most fairly applicable to the case under review.

Overhead Charges.

General Nature of Overhead Charges.—There are certain expenses, inseparable from the construction of any property, which are a necessary and proper part of its cost, but which are not capable of physical identification after the completion of construction work. These expenses cannot be covered in the estimate of "Cost of Reproduction" by the application of specific unit prices; from their nature they attach to the whole or large parts of the property rather than to any particular units. These are called "Overhead Charges".

If the reproduction estimate is to be complete, and is to include allowances for all the labor that was performed, all services that were rendered, and all expenses that were incurred in the construction of the property under investigation, it is clear that a proper allowance, in some form, must be made for these expenditures.

Recognition of Overhead Charges by Courts and Commissions.— Overhead charges in one form or another have been allowed in all past valuations. Commissions have recognized their propriety, and Courts have endorsed their allowance.

One of the most recent cases involving the allowance was the Des Moines Gas Company case. A part of the decision (238 U. S., 153) is quoted as supporting the argument of the Committee that these allowances should be made in amount sufficient to cover all real elements of overhead costs, and no more.

"What is called in this summary 'the present value of physical property', the report shows was arrived at by the Master in the manner following:

"He first found what he thought was the base value of the property, *i. e.*, 'what it would cost to produce it at the present time new, without adding thereto any overhead charges'. This figure he fixed at $1 975 026. To this he added overhead charges, 15 per cent., $296 254. From this he deducted depreciation, $333 878, leaving as the value of the property thus ascertained, $1 937 402."

Mr. Justice Day further said:

"These items of expense in development are often called overhead charges, for which, as we have already seen, the Master allowed 15 per cent. upon the base value (exclusive of real estate), or $296 254, in addition to his allowance of $6 923 for organization expenses. Of these charges the Master said:

" 'In reaching the physical value of the plant in question by the process of reproduction, it is necessary to bear in mind that the present value thereof represents much more than the machinery therein, the labor of installing and constructing them, and putting them in place to perform their various functions, ready for the manufacture and distribution of gas to its customers. Were the City of Des Moines without such a plant, and such a one as the Complainant now owns was proposed, it would be found that much more than the mere cost of labor and material would be expended. Such expenditures are termed overhead charges, and are as follows':

" '1. Time and money expended in the promotion of the enterprise, in the organization of the company and interesting capital therein, including, also, legal expenses, obtaining the necessary franchise, as well as the costs of incorporating the company.'

" '2. Then a competent engineer must be employed to prepare the plans and specifications for the plant, and make the necessary surveys, and when the work began, to superintend the construction thereof, and see that it is done properly and according to plans and specifica-

tions. The successful operation of the plant depends largely upon its proper construction.'

" '3. Then losses arising from accidents and injuries to workmen as well as the material during its construction, which is such an amount as the cost of insuring against such losses, which is between 1 and 2 per cent.'

" '4. Contingencies are such expenditures as arise from the lack of foresight and care in preparing the plans and specifications. No matter how carefully the engineer may prepare them, such expenditures invariably arise. Mr. Alvord testified that his allowance therefor would depend very much upon his knowledge of the engineer who prepared them, but that no matter who prepared them, they would invariably occur, and an allowance should be made therefor. The careful and thorough inventory in this case reduces very greatly the allowance therefor.'

" '5. The cost of administration, which includes the time and money expended by the parties who are engaged in the enterprise, purchasing the material, procuring the money for their payment as needed, and generally superintending the entire enterprise during the construction of the plant.'

" '6. It is estimated that it would take three years to complete the plant in question, and that at least one-half the time and money invested therein would give no return, and that a loss of interest would result therefrom and that such loss would be included in the overhead charges.'

" '7. Taxes during the construction.'

" 'The latter is regarded by me as very questionable. It is in a certain sense making taxes an asset rather than a liability, and the amount is so vague and uncertain that it has been given very little consideration and weight in fixing the overhead charges. Either the money or the property should pay taxes.'

" 'It must be borne in mind that these expenditures are all made during the promotion and construction of the plant, and are necessarily a part of the cost thereof. No overhead charges that do not inhere in and add to the cost thereof, should be allowed as a part of its physical value. It is not a question of what was actually expended therefor in the plant in question, but what would it cost to reproduce a similar plant at the present time.' "

In the actual construction of any property the expenses properly includible under "Overhead Charges" are actual physical costs of construction. They are in no sense intangible or non-physical values; they are not to be confused with development expense, as defined by the Committee, or going value; and must be set up as part of the physical property itself.

The late Judge R. W. Taylor, acting as arbitrator in the Cleveland Street Railway franchise hearing, said:

"Overhead charges as I have used that term and applied it to this situation means only those things for which money has been spent in the necessary work of constructing the property and putting it

into operation * * *. What I want to emphasize is that every dollar of overhead charge which is allowed for by me is a dollar that is necessarily spent in the production of the physical property."

What is true in this respect of an actual construction is also true of a reproduction estimate.

Expenses to be Provided for:

Among the expenditures which must be provided for, and classed as overhead charges are:

(*a*)—Cost of promotion;

(*b*)—Cost of financing and securing the necessary capital with which to carry out the enterprise;

(*c*)—Cost of organization, including the incorporation and organization of the company, securing of franchises, and other like expenditures;

(*d*)—Engineering, including the making of the preliminary investigations and plans, plans for the construction of the entire property, the engineering supervision of all construction and other work involved in the development of a property, except such direct supervision as may properly be included in the unit prices of various property units, or as specific charge against some particular schedule or group of units;

(*e*)—Administration, including salaries for general officers, agents, accountants, clerks, and other assistants, not included in the engineering and legal departments, and all administration expenses;

(*f*)—Legal, including salaries and expenses of law officials and costs of litigation which, depending on the character of the property and its location, may be a comparatively minor item or a very large one;

(*g*)—Interest during the period of construction on money borrowed, or on money invested in the property by its owners;

(*h*)—Taxes and insurance during construction;

(*i*)—Contingencies representing expenditures which cannot be foreseen but which from one cause or another are always of considerable size in the construction or reproduction cost estimate of any great enterprise.

These expenditures, and in some cases others of like character, are all involved in greater or less amounts in the construction of every property. They are represented by no physical property item which can be weighed or measured. They attach to the property as a whole.

Reasonable amounts can be determined for each property by a study of the history and accounting records of that property and

of other similar properties similarly located and built under like conditions.

It is a fundamental principle of valuation recognized by the Courts and commissions, and accepted by this Committee, that under normal circumstances the cost of recently created property furnishes the best basis for determining the reproduction cost of such property.

The Committee recognizes the value of the study of overhead expenses, incurred in connection with the construction of properties similar to the one under consideration, but it does not endorse the inclusion of any allowance which does not represent a legitimate expenditure of money or service in connection with the creation of the property under valuation, merely because such expenditures have been incurred for other properties. It does, however, endorse the inclusion of sufficient allowances to cover all legitimate expenditures, and it favors the use of the percentage on physical costs or any other method that will give fair and correct allowances.

See also the discussion under the caption "Shall Present or Original Physical Conditions Govern" (pp. 1363 to 1370).

Auxiliary or Collateral Costs.—Some engineers, particularly on the Pacific Slope, where large public works are more often executed by day labor than by contract, are in the habit of keeping their cost record data, and hence of estimating the cost of work, by including the so-called "Auxiliary" or "Collateral Costs", comprising incidental road or railroad construction and maintenance during the execution of the work, machinery and plant required, and various other similar classes of expenditures incident to the building of the works, in a percentage, applied to the basic material and labor costs involved in the various units or classes of work. Thus, for instance, though the labor, cement, sand, and gravel costs would be included by these engineers in the unit cost per yard for concrete, the cost, rental, or maintenance of the machinery used in the mixing of the concrete, the cost of the construction and scrapping after the completion of the work of the bins, railroad tracks, towers or travelers of one kind or another, used in the mixing, delivery, and placing of the concrete, would be included in the percentage applied to the above stated unit costs to cover so-called "Auxiliary" or "Collateral Costs". In this report it is assumed that the auxiliary and collateral costs are included in the basic unit prices.

Character of the Items Classed as Overhead Charges.—It is deemed proper by the Committee to present the considerations which prevail in the allowance of various items of overhead charges, some examples of actual costs and allowances under each head, and some of the more authoritative decisions approving such allowances.

(*a*)—*The Cost of Promotion.*—The period prior to the commencement of actual construction of physical properties is, in the case of

most properties, divided into two parts: the period of promotion and the period of corporate organization after the promotion has been brought to a successful issue.

It is clear that the amount of money actually spent in promotion may have a wide range. In the case of such properties as hydro-electric power developments, years of time and large sums of money may be required to secure title to lands and flowages and to bring the project to such form as to justify the attempt to raise funds for actual construction. Other properties may involve but little expense or but little outlay or money. Still other properties may have a large or small outlay on the initial construction, but the major part of the property, developing after operation of the pioneer lines, and built as additions to an existing operating property, may have very little actual cost of promotion.

The argument supporting the allowance of cost of promotion is ably stated in the decision of the Michigan Railroad Commission in the hearing on the application of the Northern Michigan Power Company for authority to issue securities. The decision, written by Commissioner Lawton T. Hemans, discusses "Cost of Promotion", in connection with the cost of acquisition of lands for hydraulic purposes, as follows (19 A. T. and T. Co. Com. L-253-254, June 11th, 1913):

"Another item which it seems to the Commission is properly included in the value of lands for hydraulic purposes, under the Commission's general designation of 'reasonable compensation for the time, energy and ability bestowed in their acquisition', is the item quite generally denominated 'promoter's profit', but which this Commission believes would be more truly descriptive if denominated 'cost of promotion'. The man who devotes his genius to enlisting support for great enterprises of public benefit, which his clearer foresight and keener vision has first perceived in the great world of material development, has performed services quite as valuable to the public as the engineer who later makes computations, plans and specifications, or the man who in any other position contributes to the creation of the utility.

"In passing upon the value of services of this character, the New York Public Service Commission has said that such services 'should be fairly and even liberally rewarded by the public which receives the benefit of these works. Such rewards, however, should be put upon a clear basis of business principles, should be of sufficient magnitude to encourage rather than discourage enterprise, and should not be so great as to make an exorbitant demand which is perpetual in its nature upon the community to be served. They are to be treated simply as just payments for services performed for the corporation, which services are valuable and in many cases even indispensable. Such services should be paid for upon the basis of what they are fairly worth, having regard to all the circumstances of the case'. (*In re* application, Rochester, Corning, Elmira Traction Co., 1 P. S. C., 2d D., N. Y., 166.)

"In the above case the award was made upon the basis of five per cent. on the entire cost of the enterprise, which had a total cost approximating seven million dollars. Other awards have been made by the New York Commission of varying percentages upon the estimated cost of the particular utility.

"In the matter under consideration, request is made to include in capitalization as compensation for promotion, two and one-half per cent. on the estimated cost. The Commission is persuaded that in cases of this character the item of cost of promotion attaches peculiarly to the lands and flowage rights of the development, for, if it is to be justified, as we have indicated our belief it should be, as compensation for a peculiar service incident to every comprehensive scheme of material development, then it should in principle inure to the value of the thing in relation to which the particular service was rendered. It should be compensation to the pioneer, rather than to those who claim neither originality of conception or to have assumed any of the risks of development."

That this element of cost is recognized by the Interstate Commerce Commission is shown by the provision, in the account designated as "Organization Expenses" for

"* * * Cost of preparing and distributing prospectuses; cost of soliciting subscriptions for stock; cash fees paid to promoters, and the actual cash value (at the time of the organization) of securities paid to promoters for their service in organizing the enterprise."

Examples of actual costs of promotion are difficult to cite, for the reason that the accounting for such costs has been as a rule under the head of general expense, or of organization, and have been merged with other costs which belong properly in organization, administration, or legal expenses.

(*b*)—*The Cost of Financing.*—When the project of the promoter has been investigated and found to be sound, and after a plan for the financing of the company has been adopted, there is incurred considerable expense in connection with the issue and sale of stocks or the issuing and marketing of bonds and the payment of commissions to brokers. There can be no question as to the necessity for such expenditures. They are unavoidable, and should be allowed in proper amount.

Whether discount on securities is an allowable item, is an open question. Discount may be a partial capitalization of the commercial risk had in making the investment, it may be an indication of lack of credit on the part of the company, or it may indicate that the interest rate on the bonds is lower than the conditions of the market would warrant, and the discount is in effect an adjustment of the interest rate or a prepayment of interest. If the company secures the benefit of the low interest rate, it would be unfair to capitalize the discount. Nor would it seem any more fair to capitalize excessive risk or poor credit.

That regulating authorities and Courts are not in perfect accord, is shown by the conflicting opinions quoted.

In a Montana rate case (198 Fed., 1003), Judge Hunt says:

"The master allowed 15 per cent. of $562 715.89, or $84 407.38, as a necessary and usual item of cost of reproduction. There was no evidence offered on behalf of the Railroad Commission tending to dispute the conditions which the witnesses for the complainant said existed generally throughout investing communities, namely, that a railroad, such as the one under investigation, is only able to make its financial arrangements by regarding as a part of the construction cost to which it is subjected a discount representing the difference between the amount derived from the sale of its bonds and the amount which the bonds must eventually cost the company. Recognition of discounts on securities, based upon the considerations just expressed, has been made by the courts. Of course there never could be any allowance whereby a corporation can be allowed to capitalize its own lack of credit; but where the bonds are sold at a reasonable discount, and bear a low rate of interest, it would seem to be the equivalent of selling the bonds at par with a high rate of interest. Here the 15 per cent. seems to be reasonable, the testimony showing that upon such a discount the bonds are put upon an equality in marketable conditions with the bonds of some of the very largest and most successful railroads in the country."

Commissioner Hemans, in the Northern Michigan Power Company case, says:

"The application filed herein makes request for the allowance of an item of 2½ per cent. on the cash cost of construction and physical properties to cover the item of banking and brokerage. Unquestionably every comprehensive development of the character being considered finds it expedient to enlist the services of a reputable broker who must be compensated for the investigation he shall make into the basis of the securities, which, if found satisfactory and desirable, he shall recommend to his clientage, but it is a service that is incidental to the sale of the securities or the borrowing of capital. It is an item most intimately connected with bank discount which is in part the rate which a given concern must pay for its loans. We believe that under prevailing practices, in view of the discount at which it is desired that the bonds to be authorized herein may be sold and at which like securities are sold, there is no justification for the inclusion of this item in capitalization, but that it should be included with the bond discount and ultimately extinguished by amortization from the profits of operation."

In the Des Moines gas case (238 U. S., 153), already quoted, Mr. Justice Day quotes the Master, as follows:

"Time and money expended in the promotion of the enterprise in the organization of the company and interesting capital therein, including also legal expenses, obtaining the necessary franchise as well as the costs of incorporating the company."

The Committee is of the opinion that the cost of financing should include all fair costs for interesting capital and issuing and marketing securities. Discount on securities should not be included in capital, but should be amortized over the period of the life of the securities. The fair cost of financing will depend on the probable rate of return, the credit of the promoters, and the degree of hazard attending the project. If a sufficiently high rate of return is assured and the credit of the promoters is good, the cost of financing should be low, little more than an ordinary brokerage rate. Poor credit should not be considered in estimating cost of reproduction, but probable rate of return and the hazard of the enterprise should be considered, and the fair cost of financing estimated as affected thereby.

(*c*)—*The Cost of Organization.*—Under this caption is included a class of expenditures common to all properties, distinct from "Cost of Promotion" in that they usually follow the promotion, and are incurred after the project is assured. There may be many cases in which they merge with the cost of promotion. There are undoubtedly some cases where "Cost of Promotion", "Cost of Financing", and "Cost of Organization" may all be includible under the head of organization. The class of expenses specifically referred to includes costs connected with incorporation and securing a charter, cost of securing franchise, local grants, consents of property owners, consents of the Government, as in case of construction over or under navigable waterways, and all other expenses connected with the organization of the company which is to build the property, and the securing of all the necessary rights, grants, and privileges necessary to permit it to proceed with actual construction.

In Bonbright *vs.* Geary (210 Fed., 44), already quoted, there were included "The legal expenses of organization."

In the Des Moines gas case (236 U. S., 153) there were included "legal expenses obtaining the necessary franchise, as well as the costs of incorporating the company."

In the Knoxville case (212 U. S., 1) the sum of $10 000 was allowed for "Organization, Promotion, etc." The Court assumed this to be a proper allowance without deciding.

(*d*)—*Engineering.*—As an "Overhead Charge" there should be included all engineering costs not capable of specific assignment to property units or special parts of the property.

The general engineering charge, in the case of most properties, is a very considerable item, in some cases amounting to a large percentage. The percentage differs with the character of the works and with the care and skill exercised in their construction.

In the case of a railroad property, general engineering includible as an "Overhead Charge" would cover all the reconnaissance or general investigation of the territory through which the line was to run,

the preliminary surveys, the final location, the general supervision of the chief engineer and assistants whose time is devoted to general, not specific, work, all services of consulting engineers not capable of definite assignment to specific units of property, and much of the expense of the office staff of the chief engineer's office.

In the case of most old properties, it is difficult to draw exact lines between these general engineering expenses and the engineering and inspection during construction, and in many instances it may be desirable to include all engineering as an overhead charge.

In connection with a water-works property, large expenditures must be made for investigations of drainage areas, general surveys, plans and specifications, and general supervision of all construction.

In the illustrations cited of actual engineering costs, no attempt has been made to segregate general engineering charges and direct supervision, all preliminary, general and direct charges being lumped. In case the segregation can be made, it will be found in every case that the overhead charge for general engineering is a very considerable item, which cannot be overlooked or dismissed because of charges to direct supervision.

Examples of actual costs of engineering indicate that there is a rather wide range in the percentage, even on the same kinds of property, due to the presence or absence of a large number of property units which carry high costs of direct supervision. This goes far toward supporting the argument of the Committee that these figures should be analyzed.

The construction of an important dam or aqueduct, built in place and requiring skill in designing, and a careful inspection of every part of the work as it is built, requires a larger expenditure for engineering than a large cast-iron pipe line, where the cost of laying the pipes in a trench is but a small percentage of the total cost of the line, and the work progresses so rapidly that the inspection cost is small in proportion to the total cost. The charge to direct supervision would be much greater in one case than in the other, and if such charge be made, general engineering overhead charge will be more nearly comparable with other like properties.

The cost of engineering varies, not only with the class of work, but with the character of the design and method of its execution—whether by day labor or contract, and in the latter case the nature of the contract. Works may be built with little inspection, from crude designs prepared by unskillful engineers, with the result that the cost of works may be large, although the percentage paid for engineering may be small. Works skillfully designed and efficiently constructed imply a larger cost for engineering, which should be recognized in any valuation when the works give evidence of such skill and efficiency.

(*e*)—*Administration.*—Under this head should be included all allowances to cover the salaries and expenses of executive officers during the period of construction. Also all clerks and other employees, office rent, and expenses in connection with the supervising, accounting, and other offices, except law or engineering.

This allowance should cover, when necessary, all general police and sanitary department employees.

These expenses, like those for engineering, vary, depending on the character and location of the work, legal requirements, and methods of construction followed.

As accounting records have been kept in the past, they are difficult of exact analysis by examination of old records of construction, as they have often been merged with other accounts. There has been such a universal recognition of the propriety of the inclusion of an allowance in proper amount that no argument is considered necessary.

(*f*)—*Legal Expense.*—The law expense incident to construction will vary greatly, depending on the character of the work, the population in the territory in which the work is being done, and the laws which control the operations of corporations as to securing rights or as to construction.

One property may be situated so that few if any obstructions are placed in the way of its construction, and another similar property, in another State, may have not only to pay large sums for legal services necessary to carry out the requirements of statutes and ordinances, but may have to resort to extended litigation before the work is completed. This fact makes necessary a consideration of the history of the property in connection with the allowance for "Legal Expense". This allowance has been made in nearly all past valuations. It is a proper allowance, and should be made in such amount as will adequately cover the expense in each case.

(*g*)—*Interest During Construction.*—Before the construction of works of magnitude can be undertaken, financial arrangements must be made for the advance of the necessary funds. If the period of construction involved be comparatively short, such as one or two years, it has generally been found advantageous, if not necessary, to borrow the entire sum needed in advance of construction. If, on the other hand, the period of construction is a long one, such as from 5 to 10 years, as is true in the case of most large properties, or large parts of properties which have been built during one construction period, instead of developing from small beginnings, it is generally found advantageous to make arrangements for the advance of stated sums at different times during the construction period. Generally, bankers and underwriting syndicates prefer, however, to make arrangements on the basis of purchase of the entire bond issue, though the bonds be delivered at stated intervals in blocks, and allowance be

made for repayment of a nominal rate of interest, such as 2% on unexpended bank balances.

Necessary financial arrangements involve the payment during the construction period of interest on the funds borrowed, or advanced by the owner of the property. It is logical to include these interest payments in the construction cost of the property up to the time when the workable units of the plant are put into actual revenue-producing operation; then the interest payments involved by the investment in the unit of operating property should be accounted in development-expense during the period involved in the acquisition or establishment of the business to the point where the earnings are sufficient to meet the fixed charges, as well as the operating expenses and depreciation allowance; thereafter, the interest payments should be accounted as fixed charges, to be met from the net revenue or divisible earnings of the property.

The determination of the exact time when the different workable units of the property may fairly be assumed to have crossed the dividing line between the "construction-period" and the "physical development-period" is difficult; but this is not a matter of controlling importance if proper allowance be made for the development expense element of the value in the property.

All the arguments for the inclusion of a proper allowance for "Interest" are stated so fully by the Wisconsin Railroad Commission in the Madison Gas and Electric Company case (4 Wis. R. C. R., 501-541, decided March 8th, 1910), that no further discussion appears to be necessary:

"While all these points are interesting, and contribute to an intelligent discussion and determination of the question at issue, it would seem that the interest during construction, correctly allowable in a valuation under ordinary conditions, would be interest at the current rate on the cost of each part of the plant during its construction. The element of cost by reason of interest during construction is one which cannot be escaped. It is present to some extent no matter what the method of financing the construction may be. From the time the investment for construction is made until the completion of the entire plant enables the investment to become active as an integral part of a working whole, there is the element of interest, for that investment is necessarily involved and is necessarily idle until the completion of the plant to a working point. The fact of interest, like the fact of depreciation, is present, no matter what method is employed for financing it. This is as true when the money is furnished by the owners as when it is borrowed by them. The theory upon which such interest rests is sound, and remains so even in isolated cases where the investors decide to charge no interest, and choose to donate the same to the consumers in the way of lower charges for the services rendered. Even if the company let a contract for the complete construction of a plant, to be paid for in no part until wholly

completed to the operating point, interest cost would come in as a part of the contract price, even though not expressly set forth. In that case the contractor would have to ask a higher price to cover the interest cost. Some additional compensation is needed for the investment made by him during the progress of the work and which is necessarily idle until the completion of the plant. Should the company itself construct the plant and be able to arrange to do the entire work on credit—gathering the material, machines and equipment, labor and everything additional needed on credit, to be paid for at the completion of the plant, at the point of operation—the interest cost would nevertheless enter in some form in the entire cost of construction."

It has been the practice on many valuations to assume a period of construction, and to assume that the interest during the period of construction would equal the interest for the entire period on one-half the total cost. In making the assumption as to a proper period of time in which any property could be constructed in connection with the reproduction estimate, due consideration should be given to the actual history of the development of the property, and to the sequence in which the various parts of the property were built or would be built under present-day methods, and making use of existing facilities. Allowances for interest and other overhead charges should be based on actual experience in the construction of the property under investigation, or similar new properties, as far as possible, and not on any such general assumption as stated, as such assumption is likely to give results which would be too small in many instances and excessive in others, and the attempt should be to determine a figure which will be just and equitable in each case.

Some of the earlier railroad appraisals assumed that the property would be produced in short sections, each taking one year for construction, and that an allowance of 3% on the entire cost, or an average of one-half year's interest, would be ample.

While it is undoubtedly true that some properties have been built and opened to traffic in short sections, and have thus carried a small interest charge, and some of these properties would be so built if they were to be reproduced, such an assumption is not applicable to all properties.

From the viewpoint of reproduction under present conditions, interest during construction should be based on carefully prepared construction and financial programmes which will provide for a proper period for reproduction, giving due regard to the history of the enterprise and to the periods of times required for the construction of different portions of the property, as well as a reasonable sequence in the order of reproduction, and will provide fully for all economies. The financial programme should be so adequate as to provide ample funds and materials, as needed during construction, and to eliminate

delays. The rate of interest should be based, not on the financial credit of the property at the time of appraisal, but on the assumption that the property is a new property, and will be developed during the reproduction period assumed, and giving due weight to the hazards attached to each property.

From a strictly historic point of view, the time for which interest during construction should be allowed should be determined on the basis of historic conditions, in the same way as other items entering into the estimate of cost, except that normal present conditions should be used in fixing the rate or rates of interest to be used. Due regard should be given the history of the property as to its effect upon the credit of the company, having in mind that in some existing utilities large parts have been built piecemeal after the original property had been fully established, without the degree of risk and hazard attending the building of the original property, and that in some other utilities the risks attending the development of the additions were fully as great as those attending the construction of the original property. The financial programme should be so adequate as to provide such ample funds and materials as would be needed during the construction periods of the different parts of the property to eliminate delays.

It is not intended by the above statement to suggest that it is necessary to obtain the history of each item added to the original property or used to replace an item which has reached the end of its life, because this would be impracticable and unnecessary; but that the historical basis should be used as a guiding factor in determining the period for which the interest during construction of such items or classes of items should be computed. In other words, the interest during construction allowance for items added to a property from time to time after the original construction should extend only over the period of time required to build each particular item and put it in operation, with a reasonable addition to provide for procuring the money in advance of the beginning of construction.

Interest should then be computed on the various parts of the property, taking into account the time required to construct each and the total time that must elapse from the completion of that part until the operating units of property reach the point of revenue-producing operation.

(*h*)—*Taxes.*—The money paid out for taxes on lands, buildings, materials, and equipment during the period of construction of a utility property would seem to be so obviously a proper charge to capital invested in plant as to require no supporting argument, yet it has not received the unqualified endorsement of the Courts. An illustration may be cited in the case of an electric light and power company in a large mid-western city. The rapid growth of the city and increase of the business necessitated very large increase in the plant facilities, and

it was determined to build a new power station in a new location. A tract of 22 acres was acquired in 1911, and work was at once commenced on preparation of plans. Actual construction was begun in the spring of 1912, and the plant was put in operation during the summer of 1915. During this period 3 full years taxes on the land were paid, and the increase each year in the assessments, on account of the improvements, resulted in a payment during the period of more than four times the tax that would have been paid on the basis of the assessed valuation at the time of purchase. This expense is unavoidable; it is, like interest, a charge that must be provided for, and met out of funds raised for construction. The percentage of the total construction cost may vary on different kinds of property, and care must be taken to make sufficiently full investigations to insure the accuracy of the estimate. The Committee believes that an allowance for taxes, in proper amount, is a fair charge to construction, and should always be included in the reproduction estimate.

(*i*)—*Contingencies.*—Contingencies in construction are those added costs or added items of construction which are made necessary by unforeseen difficulties, or events beyond the control of the builders, or are due to conditions which were not fully determined at the inception of the work. Similarly, the contingencies in an estimate of reproduction are those due to conditions which could not be fully determined at the time of making such estimate of cost and such conditions always prevail, in greater or less degree, depending upon the character of the property and local conditions, the age of the property, the available records, and many other factors.

Contingencies vary in amount with different kinds of property and with different property units. On classes of work in which the contingency item is likely to be large, the proper percentage to allow for contingencies will depend largely on the extent and accuracy of the investigation.

Perhaps no one item of overhead charges on past appraisals has drawn more criticism, and in some instances ridicule, than the allowance for contingencies. The Committee, at the outset of its discussion of the subject, desires to state that no allowance for contingencies is advocated that is not a reasonable one.

It must be emphasized that the allowance for contingencies is an allowance for the unseen and indeterminate cost in reproduction which could not be seen, investigated, and determined in advance of original construction, or at the date of valuation. Many people appear to be unable to realize the fact that it is not possible to see all the things that were done and know all the conditions that accompanied the doing of them when investigating an existing great public utility.

It is fully as difficult to estimate correctly the cost of reproducing an existing property as it is to estimate the cost of producing one

which does not exist, and it may be more difficult. To illustrate, assume one of the large hydro-electric developments or water-works reservoirs. The property is, let us say, 12 or 15 years old. The valuation engineer finds a massive masonry dam, some buildings and equipment capable of definite measurements, and a large lake or water covering several hundred acres. All roads, railroads, and other property surrounding the reservoir are in good condition, and their appearance would not indicate whether they were 12 or 40 years old. The plans, field notebooks, and construction records may be complete, but are more likely not to have been fully preserved, as in many properties the value of old notes and construction records has not been recognized. The problem is to estimate the cost of reproducing the property. There will be many costs which can be covered only by a contingency allowance.

The construction engineer, about to enter upon the construction of such a property, finds in the valley to be occupied all the various properties and constructions that are to be removed, he has available the results of explorations, surveys, soundings, borings, and other investigations which will determine what will be encountered. He has equal knowledge with the valuation engineer as to costs of material, labor, transportation, tools and equipment, and other things needed, and his facilities for accurate estimating are just as good. Yet, in producing the property, he encounters costs not foreseen, due to weather, floods, labor troubles, transportation difficulties, delays, and many other causes.

The contingencies cannot be listed in advance—otherwise they would not be contingencies.

Those who ridicule or omit contingent allowances, or argue that they apply to individual items rather than to the plant costs as a whole, ignore general construction experience. The allowance for contingencies is not an allowance to cover error, as has often been assumed. To some extent contingencies in both construction and reproduction are due to failure to schedule completely all work to be done, but, to a far greater extent, contingencies are due to those unseen, unavoidable, and uncontrollable causes that neither engineer, nor owner, nor contractor can foresee.

Errors of judgment may occasionally add to the cost of construction, and the Committee is of the opinion that a provision for this is justifiable.

The Committee endorses no allowance for contingencies which will add arbitrarily a percentage to a thing that is known and definite; but it does contend for an allowance in all cases except those in which it is certainly known that contingencies would not be incurred.

Character of Contingency Expenses.—Omissions from Schedule.—In both construction and reproduction estimates on large properties, where many thousands of items of property are scattered over a wide area,

or, as in the case of railroads, over hundreds of miles of territory, there are certain to be omissions, not only of minor items, but of large items of property. This applies, not only to property which is capable of easy identification, but to a greater extent to that which was built in connection with the construction, and was essential to it, but which is not in constant use, and of which ownership consequently is finally lost sight of.

Hidden Construction.—Unless the most complete records are available, much of the expense which adds to the cost of foundations of all kinds is apt to be overlooked. Pumping or coffer-dams in some localities, and quicksand or other treacherous material often increasing the size or depth of foundations, makes the cost not only greater per unit but adds largely to actual quantities in excess of the plans. Costs incident to settlement of the surface on marshes, causing a large increase in the fill required, are sometimes included in contingencies, but should be allowed for preferably in the schedule of quantities.

Losses Due to Flood, Storm, or Bad Weather.—Frequent and unavoidable expenses are those due to storms, floods, slides, and wash-outs. There is also the added expense due to freezing or wet weather. The examples of this class are very numerous; hardly any large property has been built which has not had to contend with such difficulties.

Strikes, Labor Troubles, Delays in Delivery of Materials.—Labor troubles are a frequent and expensive occurrence, and, at times, not only delay completion, but add materially to the cost. The non-delivery of material, often necessitating the purchase of small quantities at high prices, or causing expensive delays, is another class of contingency expense which is continually met. Delays are often caused by injunction, or other legal processes or failure to raise the necessary funds as needed, causing, not only extra expense on the work, but loss of interest. The failure of contractors or the abandonment of work by them, is often a serious cause of contingent expense.

Moreover, it often happens on large works that the losses due to interruption of or delay in construction are largely augmented by the loss in efficiency of the working force, incident to building up a partly new organization upon the continuation of construction. The suspension of active work upon the construction of the Los Angeles aqueduct, due to temporary lack of funds, is stated in the report to have caused an additional expense of $250 000.

Construction Outside of the Main Property.—In nearly all large properties it will be found that, either owing to the special location or conditions, or on account of relations with the public or other owners, it is frequently necessary to build considerable work which is in no wise a part of the property under investigation, but is required to readjust existing conditions to permit the work to proceed.

Illustrative Examples of Actual Contingencies.—In building an electric road near Toledo, Ohio, contracts for grading were let at 22 cents per cu. yd. for earth. One section involved a crossing under a railroad. The presence of quicksand and water in the cut made ordinary methods of handling material ineffective, and the contractor failed. The work was completed on the basis of cost plus a percentage, at a cost of 84 cents per cu. yd.

In 1895 the Ann Arbor Railroad was raising an embankment over a swamp. A "sink hole" developed which broke under a freight train, causing not only a loss of many thousands of yards of earth, but damages to property of more than $25 000.

In March, 1905, the flood conditions in Ohio and Indiana were unusually severe. An electric railway bridge over the Maumee River was under construction, and seven high concrete piers were completed ready for superstructure. An ice gorge formed about 2 miles above the bridge, and when it broke two piers were swept away, involving a contingency expense of more than 15% of the entire cost of the bridge.

The Cleveland Railway, in the construction of a new car-house, lost the entire structure in a tornado after more than 70% of the work had been completed.

Elliot Holbrook, M. Am. Soc. C. E., cites examples of contingencies, as follows:

"A piece of construction with which the writer was concerned consumed over two years in construction, while it was expected it would be completed in one season. The result was that owing to increased outlay and greater length of construction period, interest alone was increased by an amount equal to 5 per cent. of the original estimate of total construction cost. A steel bridge near Anoka, estimated to cost $21 000, actually cost $32 000, on account of floods taking the coffer-dam out. Pipe line at Dassel, Minn., estimated to cost $6 600, actually cost $8 300, owing to delays that carried the work into winter, necessitating excavation in frozen ground. Bank widenings in Dakota, estimated to cost $15 000, actually cost $27 000, owing to excessive rains. Change of line from Teton, Mont., to Tunis, Mont., estimated to cost $10 500, actually cost $33 000, on account of a hill giving way and burying a steam shovel. A change of grade, estimated to cost $7 100, actually cost $12 000, owing to slides and wet conditions of material. New yard at Blair, Mont., estimated to cost $41 500, cost $66 700, owing to heavy storms washing out grading and causing delay. Drainage tunnel at Alvin, estimated to cost $16 700, actually cost $23 000, on account of trouble encountered with water. In fact, in the construction of a railroad, there is almost a continuous succession of unforeseen and unforeseeable difficulties that increase the time necessary for construction and the total outlay required."

In the case of subway construction now under way in the City of New York the provision originally made for contingencies, 5%,

has been found to be inadequate, and additional elements of cost have so far developed to indicate that they will amount to upward of 14%, exclusive of the provision that may have to be made for contractor's claims, and further charges for interest on money during construction caused by the non-synchronizing of completed portions of the system.

Many other examples have been cited to the Committee. It is believed that, if it were possible to separate from the bare cost of nearly all large works those items that were actually contingencies, the list might be greatly lengthened. Attention should be directed specially to the fact that nearly all available cost records give total actual construction, but do not give original estimates or originally estimated quantities. To determine contingencies therefor, it is necessary to go in most cases to the original records and make special analyses. This will account for the comparatively small number of cases in which this item of overhead charges has actually been segregated.

The close estimation of cost is an art, of which comparatively few men are masters. In its final analysis it is a matter of judgment, founded upon knowledge and breadth of experience in engineering construction, and discrimination in the determination of the relative effect of different conditions and factors entering into the problem, as well as in the application of well kept records of previously built works. Synthetic cost analysis has some advantages, if limited to the main and readily determinable cost influencing items, though it has the grave danger of implying a degree of accuracy in the results obtained, not actually realized, and the tendency to minimize and under-estimate the elements of hazard and uncertainty, which always exist to a greater or less degree, on any large construction work. Direct comparison with costs of other works, of similar character, under similar conditions, when possible, furnishes the most reliable guide. The method of always comparing the cost of different classes of work, on broad lines, with a previously determined "average-scale", based on general experience under more or less similar conditions, and getting the coefficient or factor, as compared with the average-scale, of each work, is most helpful, particularly to the young engineer.

Allen Hazen, M. Am. Soc. C. E., has discussed this subject in an admirable article on "The Art of Making Rapid and Reliable Preliminary Estimates of Cost",* from which the following paragraphs have been abstracted.

Extra Cost of Novel Designs.—Work on novel designs commonly costs more than work following standard designs. This is true even when well tried methods are first introduced in new places. Such work may be too small to attract bidders from a distance. The unit cost will then overrun anticipated prices. An under-estimate of cost is

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frequently made on work because the estimator fails to realize what a great effect familiarity with the methods of performing work has upon the cost. To realize this one has but to think of the difference between present methods and costs of building tunnels and subways and deep foundations, and the methods and costs of only 10 or 20 years ago. Not only is the risk which the contractor takes now less, but methods which have been thoroughly tried out are at his disposal as well as experienced foremen and laborers, to do the work more economically. For a structure of new or novel design much caution in using prices that may be standard on other kinds of work must be used.

Small Jobs Cost more in Proportion than Large Ones.—Another common cause of under-estimating costs is the use of figures on large pieces of work for estimating small work. The engineer often overlooks the large cost of overhead charges, the waste of labor and the cost of plant caused by organizing a force to perform a small piece of work. He too often forgets that for small work the work must be done by less efficient methods, or the cost of plant prohibits the use of expensive machines. Perhaps the most common case of such underestimating is where the job, although a large one in the aggregate, consists of many small pieces of work of widely varied character. Such a job is troublesome and costly for the contractor, and the experienced man knows it and puts in a corresponding bid.

It must be remembered that bids follow the law of supply and demand; that general slackness in construction work calls out bidders and low prices. The condition of the money market, the cost of materials, and the general opinion of the condition of contract work are of course matters of much importance. It seems scarcely necessary to say that the engineer should know where and how the contractor is to obtain his materials and have a fairly definite idea of the cost, either in dollars and cents or as a comparative figure to other work.

The engineer in estimating should try to look at the work from the standpoint of the contractor, should try to remember that no work runs as smoothly along as the contractor wishes, that labor conditions and other matters often spoil the best laid plans. He should endeavor to keep in mind the various work he has watched or performed and the numerous times when unexpected conditions added largely to the cost.

Generally speaking, estimates on proposed work by men of limited experience are too low, yet it is not unusual for an engineer to be so impressed with the difficulty of a piece of work that he over-estimates the actual cost, and it is more common to find that he has over-estimated the bids of the contractors.

It is of much value to have two methods of arriving at estimates. If an independent check method can be made, even though a rough

one, a failure of the two results to agree often leads to the discovery of serious errors in the application of one or the other method.

Conditions of Success in Estimating.—One of the first requisites for successful estimating is a fair and unprejudiced, and moderately pessimistic mind. The estimator must be alert for new and cheaper processes and methods. and conservatively sceptical concerning their merits. Moreover a successful estimator must have had experience in actual construction. As a general rule an engineer should not make an estimate for a structure that he would not know how to build.

Next, as a requisite to successful estimating, may be mentioned a broad basis of cost records of actual work, more or less similar to that for which estimates are to be made. A good and safe method of using the cost data and adjusting it for application to new conditions is equally important.

And, finally, the estimator must have the will to refuse to make estimates for work that he does not understand.

CHAPTER VI.

DEPRECIATION AND APPRECIATION.

Depreciation in Valuation.—Perhaps there is no single subject in connection with valuation that has caused more trouble than depreciation. This has been due to various causes, perhaps not the least of which has been confusion in the use of the term. Depreciation is sometimes used to mean decretion, which is loss of service life, sometimes to mean the money allowance made in the bookkeeping to offset accruing loss of service life, and sometimes the loss of value existing at any time due to the loss of service life or any other cause. The Committee will use it only as meaning the loss of value or worth of property units which are parts of going concerns. Although this may be due to many causes, the general discussion will include consideration only of those effects which, like wear and tear, age, use, and obsolescence, or inadequacy, bring a physical property unit gradually to the end of its service life.

Another difficulty has been the failure of owners and Courts to comprehend depreciation. Neither of these have realized in the past the necessity for and equity of earning and setting aside allowances to provide for accruing loss of capital due to the accruing, but not yet total, loss of service life of the parts of a property. Nor is this difficulty wholly removed at the present time. In the earliest decisions of the United States Courts bearing on this subject (Union Pacific R. R. Co. *v.* United States, 99 U. S., 402, and United States *v.* Kansas Pacific Ry. Co., 99 U. S., 455, both decided October, 1878), it was held: in the first case, that all expenses for maintenance of the property should be included in operating expense, only cost of original construction and subsequent enlargement and improvement of the works being capital expenditures; and it was even held that ordinary betterments wisely might be made from earnings, apparently without a charge to capital, a practice now generally disapproved by Courts and commissions; and in the second case, that no allowance for depreciation that does not represent cash actually spent can be included in operating expense.

Nineteen years later the Supreme Court of California (in San Diego Water Co. *v.* San Diego, 118 Cal., 556, 1897), mentions depreciation, and in the prevailing opinion given by Justice Van Fleet, reinforced by the concurring opinion of Justice Garouette, distinctly holds that an allowance should not be collected from the rate-payers for it, the concurring opinion saying:

"But such a thing is all wrong, for it results in the consumers of water buying the plant and paying for it in installments."

But it is fair to say that this concurring opinion is based on the notion that such things as tunnels, wells, reservoirs, water rights, and real estate suffer no depreciation;

"* * * there is no wear and tear, no permanent and gradual destruction by use and age. Most of them stand everlasting as the hills."

and that it further admits that the replacement of worn out items is a proper charge to operating expenses, saying:

"If improvements are made in the plant as it stands, as, for example, a new pipe substituted for an old piece of the same size and quality, such charge should be considered operating expenses."

which seems clearly to uphold what will be described later as the replacement method of maintaining property that is supposed to wear out. The decision makes the mistake of finding that certain items of property which do go out of service for one reason or another are "everlasting as the hills."

It is interesting to note that there was a dissenting opinion, handed down by Chief Justice Beatty, which says that after including as current expenses all operating expenses reasonably and properly incurred, taxes, and the cost of current repairs, there should be added to the sum to be collected annually an allowance for a sinking fund sufficient to replace any parts of the plant which, at the end of a term of years, will be so worn out as to require restoration, when this restoration occurs.

The foregoing is introduced because of the appearance, apparently for the first time, of a discussion of the propriety of collecting and setting aside a depreciation allowance, and particularly the approval of such a course as given in the dissenting opinion. A year later, the same Court (in Redlands, etc., Water Co. *v.* Redlands, 212 Cal., 312, 1898) said:

"In a municipal ordinance fixing rates * * * the water company is not entitled to be reimbursed from the income derived from rates, fixed by the ordinance, for interest upon its indebtedness, nor for depreciation of its plant, aside from the amount requisite for its maintenance and repairs during the year."

This decision was consistent with that in the San Diego case, and referred to it, and is here quoted because the United States Supreme Court (in San Diego Land and Town Co. *v.* Jasper, 189 U. S., 439, 1903) referred to this case with approval. And this was as late as 1903. The language used at page 446 of the report is as follows:

"We will say a word about the opposite contention of the appellant that there should have been an allowance for depreciation over and above an allowance for repairs. From a constitutional point of view we see no sufficient evidence that the allowance of six per cent. on the value set by the supervisors, in addition to what was allowed for

repairs, is confiscatory. On the other hand, if the claim is made under the statute, although that would be no ground for bringing the case to this court, it has been decided by the Supreme Court of California that the statute warrants no such claim."

Four years later, at least one Court had seen a light, for (in Perkins *v.* Northern Pacific Ry., 155 Fed., 445, 1907) Judge Lochran, of the Circuit Court of Minnesota, states, among other things:

"It is evident * * * that a railroad, like everything else, will wear out in time, and they have been used so long in this country that there can be a reasonable estimate of the percentage of loss each year from depreciation of the roadbed, culverts, bridges, and rolling stock; that it would be proper to lay aside a reasonable amount to furnish replacements, renewals, and repairs when needed; and that if that was not done the railroad company might soon be in a position in which it could not keep up, with the rates that it was getting, and maintain its property in an efficient state to render such service as the public is entitled to receive from it. Now this is a matter in which the public has an interest as well as the railroad companies and the stockholders of the railroad companies."

"There is a danger that this feeling of selfishness may lead them too far and reduce this compensation so much that it will not enable the railroads to serve them with efficiency—to keep up their roadbed, culverts, bridges, and everything so that they will be entirely safe for the transportation of passengers and freight, and to keep the rolling stock in the best state of efficiency and enable them to provide the best service attainable, and that is exactly what those corporations are required to do. They are required to exercise the highest degree of care in relation to the transportation of passengers, and a high degree of care in relation to the transportation of freight, and it is certainly for the interests of the people that they should be enabled to do this; and it would be a very short sighted policy which would reduce the compensation of these railroads to a degree that would disable them from performing these services fully and fairly for the benefit of the people."

It was not until 1909, however, that the Supreme Court of the United States recognized the character of depreciation and the necessity for collecting an allowance to cover it as it accrues. This recognition appears in the decision handed down by Mr. Justice Moody, in what has come to be known as the Knoxville case. (City of Knoxville *v.* Knoxville Water Co., 212 U. S., 1, decided January 4th, 1909.)

This decision, here quoted somewhat at length, holds that, not only is it proper to set aside periodically an allowance to cover the accruing loss of value of aging property, but that it is the plain duty of operating companies so to do and to collect, from the public served, enough to maintain at all times the value of the investment unimpaired.

It also holds that though the cost of reproduction is one way of ascertaining the present value of a plant, like that of a water company, which has been in use for many years, the test would lead to obviously

incorrect results if the cost of reproduction is not diminished by the depreciation which has come from age and use.

"The cost of reproduction is one way of ascertaining the present value of a plant like that of a water company, but that test would lead to obviously incorrect results, if the cost of reproduction is not diminished by the depreciation which has come from age and use. The company contends that the master, in fixing upon the valuation of the tangible property, did make an allowance for depreciation, but we are unable to agree to this. The master nowhere says that he made allowance for depreciation, and the language of his report is inconsistent with such a reduction. The figures which he adopts are those of a 'fair contractor's price.' The basis of his calculation was the testimony of an opinion witness called from the company. That witness submitted a table, which avowedly showed the cost of reproduction, without allowance for depreciation. The values testified to by him were adopted by the master in the great majority of cases. The witness's valuation of the tangible property was somewhat reduced by the master, but the reductions were not based on the theory of depreciation, but upon a difference of opinion as to reproduction cost.

"The cost of reproduction is not always a fair measure of the present value of a plant which has been in use for many years. The items composing the plant depreciate in value from year to year in varying degree. Some pieces of property, like real estate for instance, depreciate not at all, and sometimes, on the other hand, appreciate in value. But the reservoirs, the mains, the service pipes, structures upon real estate, standpipes, pumps, boilers, meters, tools and appliances of every kind begin to depreciate with more or less rapidity from the moment of their first use. It is not easy to fix at any given time the amount of depreciation of a plant whose component parts are of different ages with different expectation of life. But it is clear that some substantial allowance for depreciation ought to have been made in this case. The officers of the company, *alio intuituo,* estimated what they called 'incomplete depreciation' of this plant (which we understand to be the depreciation of the surviving parts of it still in use) at $77 000 which is 14 per cent. of the master's appraisement of the tangible property.

"A witness called by the city placed the reproduction value of the tangible property at $363 000, and estimated the allowance that should be made for depreciation at $118 000, or 32 per cent. In the view we take of the case, it is not necessary that we should undertake the difficult task of determining exactly how much the master's valuation of the tangible property ought to have been diminished by the depreciation which that property had undergone. It is enough to say that there should have been a considerable diminution, sufficient at least to raise the net income found by the court above 6 per cent. upon the whole valuation thus diminished.

"The company's original case was based upon an elaborate analysis of the cost of construction. To arrive at the present value of the plant, large deductions were made on account of the depreciation. This depreciation was divided into complete depreciation and incomplete depreciation. The complete depreciation represented that part

of the original plant which through destruction or obsolescence had actually perished as useful property. The incomplete depreciation represented the impairment in value of the parts of the plant which remained in existence and were continued in use. It was urgently contended that, in fixing upon a reasonable return, the amounts of complete and incomplete depreciation should be added to the present value of the surviving parts. The court refused to approve this method, and we think properly refused. A water plant, with all its additions, begins to depreciate in value from the moment of its use. Before coming to the question of profit at all, the company is entitled to earn a sufficient sum annually to provide, not only for current repairs, but for making good the depreciation and replacing the parts of the property when they come to the end of their life. The company is not bound to see its property gradually waste, without making provision out of earnings for its replacement. It is entitled to see that from earnings the value of the property invested is kept unimpaired, so that at the end of any given term of years the original investment remains as it was at the beginning. It is not only the right of the company to make such a provision, but it is its duty to its bond and stockholders, and, in case of a public service corporation at least, its plain duty to the public. If a different course were pursued, the only method of providing for replacement of property which has ceased to be useful would be the investment of new capital and the issue of new bonds or stocks. This course would lead to a constantly increasing variance between present value and bond and stock capitalization—a tendency which would inevitably lead to disaster, either to the stockholders or to the public, or both. If, however, a company fails to perform this plain duty and to exact sufficient returns to keep the investment unimpaired, whether this is the result of unwarranted dividends upon over-issues of securities, or of omission to exact proper prices for the output, the fault is its own. When, therefore, a public regulation of its prices comes under question, the true value of the property then employed for the purpose of earning a return cannot be enhanced by a consideration of the errors in management which have been committed in the past."

This decision has been supported in all subsequent decisions of the United States Supreme Court having to do with the matter, and therefore it stands to-day the law in this country. Questions of interpretation have been raised by public utility owners, lower Courts have not all followed the decision literally, and there are certain practical questions arising in connection with the determination of depreciation in the valuation of a property, all of which require careful consideration and will be discussed in this chapter.

Depreciation being defined primarily as loss of value or worth, the depreciation that is of concern in valuation proceedings is that which is measured by the sum that should be deducted from original cost to date or from estimated cost of reproduction new, as a step in finding that which the Courts have called "fair value." The Committee will call this the "depreciation of valuation", or "fair depreciation".

Three Aspects.

The Committee will consider depreciation from three aspects:

1.—The cause—*decretion or loss of service life* of physical property;
2.—The record—*accounting depreciation;*
3.—The amount sought—the *depreciation of valuation,* or *fair depreciation.*

By "decretion" is meant that loss of service life of a physical property, or property unit, or item, which may be estimated by an inspection of the property, considering the history of its past service and the probable character of its future service, the method and character of its maintenance, and all other pertinent facts concerning it. It is the fact of loss of service life, regardless of its effect on value or anything else. Consciously or unconsciously, the result will be expressed as a fraction of the estimated total service life (elapsed life plus expectation of life) of the property, or property unit, or item, and this must be converted into a corresponding fraction of cost, in order to find the loss of value resulting from the decretion. Some minds may not consciously go through the elementary process, but will estimate at once the final money sum.

By "accounting depreciation" is meant that sum which, in the bookkeeping of the owners of the property, has been charged to operating expense and deducted from the capital, either directly or by setting up a reserve, to allow for an estimated or actual depreciation of a property unit or item. A definition conforming to methods prescribed by the Interstate Commerce Commission is the sum charged to operating expense to provide a reserve* to cover the cost less salvage of a given property when it is retired from service. Accounting depreciation may or may not be equivalent to the actual or *fair depreciation* at any time. Whether or not it is approximately equivalent, will depend on the methods used and the accuracy of the forecast of service life of the property units or items.

The "depreciation of valuation" or "fair depreciation" has been defined above.

In valuation discussions, depreciation cannot be treated wholly independently of its cause or of accounting methods, but, so far as possible, the Committee will consider: first, only decretion or the cause; second, only accounting depreciation; and, third, the depreciation of valuation.

Decretion.

Causes of Decretion.—Decretion may be due to use. A machine may have been purchased new, may have been in use a year, may show no signs of wear, and yet experience tells that the machine when new

* For a definition of this term, see p. 1460.

was probably good for only 10 years. One of these years has elapsed; the machine is as valuable in its present output of service as ever it was, but, a year of its life has gone into service, and hence, approximately one-tenth of the life of the machine must be estimated as gone, and therefore decretion exists to that extent. Examination of the particular machine may lead the examiner to estimate that it has still 11 years of service before it, in which case its total life will be estimated at 12 years, and only one-twelfth of its service life will be gone. Examination always should be made, and consideration should be given to the character of past and probable future service, and the method and character of maintenance, because, on all these will depend the estimate of total service life. Units of service may be of more consequence than years of service in estimating service life.

Decretion may be due to age alone, as some properties grow old regardless of the use to which they are put. Sometimes this aging depends on the care given the properties. Wood decays, iron oxidizes, brick disintegrates, even stone is destroyed by the elements, some kinds more rapidly than others. The particular use to which these substances are put, whether in sheltered or exposed situations, may determine the rate of decretion, which for some uses may be practically or actually nil, within the range of human estimates. The effect of weathering often may be overcome by periodic repairs, and a given structure may remain in use for many years, and even perpetually, the structure of a given day having in it no item that was in it when it was first built. Such structures will always show some physical deterioration—the immediate or approaching need of some replacements or repairs.

Decretion may be due to immediate or approaching inadequacy. A given machine may be doing a work of constantly increasing magnitude which now or later will require a larger machine before the present one is worn out. The present machine may be good for a considerable service within its capacity. It may have, therefore, a large salvage value, but, for the purpose for which it is being used, it is approaching the end of its service life.

Decretion may be due to obsolescence. A machine may belong to a class that is more or less rapidly changing patterns and forms, due to an advancing art or science. Long before it is worn out, new forms may have been devised which are so much more efficient that it may no longer pay to keep the old machine in service. Or advancing civilization, growth of population, and changing character of social life may call for a different service from that rendered with satisfaction to the people served under the old conditions. This sort of decretion is not due to inadequacy, but to obsolescence.

The several causes mentioned may exist singly or together in a given plant, or plant unit.

Decretion of a plant unit may also be caused by accident.

Decretion always Present in a Going Property.—Decretion is likely to exist in almost every physical item of a going property except land. Even the Pyramids are slowly passing away, and, between items of very long life and those, like working tools, of very short life, there are all degrees of length of service life. Some properties, like water-works or gas properties, are essentially all new when they begin to operate, because, though some pipes may have been in the ground for some time, their total service life is so long that this preliminary exposure to decay may be neglected. Shortly after a water or gas property begins to operate, its machinery and structures have begun to deteriorate, and could not be inventoried as new, even for the purpose for which they are used, although they may be giving just as efficient service as new machinery and structures could give. Other properties, like railroads, have some items that are essentially old when regular service begins. Ties, for instance, begin to decay with some rapidity as soon as they are laid in the track, and often by the time a newly created extensive property has reached the operating stage they are being replaced almost or quite at the usual normal rate on the older part of the road.

Due to these facts, no going property could ever be inventoried as new if taken item by item or unit by unit, because some of the service life of practically every item or unit will have passed. As a result of this fact it can be shown that when the properly weighted average of all the decretion of the various units of a well-maintained railroad, for instance, is found, the railroad property as a whole cannot show more than from 80 to 90% of new condition, or there is estimated, say, 15% loss of value due to decretion of various units.

The Troublesome Problem.—Although, in a well-maintained property, decretion is always present in some degree, yet in some cases this decretion, converted into loss of value, should not be considered to be a deductible quantity in finding the value of the property—that is, it should not be considered as depreciation of valuation, or fair depreciation. Unfortunately, this has not been entirely understood.

Whether or not, and to what extent, if any, the loss of value due to existing decretion shall be deducted from original cost to date or cost of reproduction new, when finding fair value for any purpose, is the great troublesome problem of depreciation in valuation. This problem will be discussed after explaining accounting depreciation.

Accounting Depreciation.

General Statement.—To understand accounting depreciation, as defined in the Glossary, it is necessary to know something of the fundamentals of methods of accounting for depreciation. Bringing

depreciation into the accounts of a company owner is one of the steps necessary to keep the earnings and property statements correct.

Many owners have not understood, and some do not now understand, the necessity for charging off depreciation as it accrues. Sooner or later, such owners are brought face to face with the necessity of renewing plant units with no means available, because profits have been over-estimated and a part of the capital has been distributed in the past by reason of the failure to account, as it progressed, for depreciation, which though not apparent, was nevertheless accruing.

Moreover, with respect to public service properties likely at some time to be valued for purchase, or the settlement of some rate or other public question by a regulating body, or a Court, the owner may find the Knoxville decision troublesome, tending to prevent a finding of the real value of his property, if he has not been careful to account for depreciation as it has accrued.

There are various methods of accounting for depreciation. In many cases the method is prescribed by a public regulating body. When it is not thus prescribed, the method applicable to any given property may depend on the law, or, with different methods conforming to the law, on fairness or convenience. What is fair or convenient may depend in part on the character of the property to be accounted for, whether made up of long- or short-lived items, whether stationary, growing rapidly or slowly. The method to be adopted for a given property should accord with a reasonable interpretation of law, and sound business principles.

Fundamental Principles.—

1.—The owner of a public utility is under obligation to the investors in its securities to maintain the integrity of the investment.

2.—The public is under obligation to the utility owner to pay a fair price for the service rendered and to meet fairly its other contractual obligations fairly. Of concern in the present discussion is the obligation to include, in the fair price for service, an allowance for the maintenance of the integrity of the investment; that is, an allowance to cover the gradual destruction or deterioration, including estimated inadequacy and obsolescence, of the physical property as this occurs. This means that when, for any ordinary reason, an item of physical property is retired, a sum equal to its cost, or some equivalent of this sum, paid by the people who have been served, should be in the hands of the company owner to make good the loss due to the retirement of the item.

It is immaterial to the maintenance of the service how this is paid by the people, whether in a lump sum when the item goes out of service, or a little at a time, more or less in proportion to the gradually lessening service life of the item; but law or convenience may dictate the

manner of payment. As a matter of convenience, and of fairness to the using public and the owner, those methods of payment should be adopted which, conforming to law, avoid great irregularities in operating expense, in dividends, or in rates for service—that is to say:

3.—The return to the investor and the rates to the consumer should be kept reasonably stable and uniform from year to year, and should be fair.

The respective obligations of the public and the corporation are set forth in numerous Court decisions; the obligations of the utility to the investors in its securities are plainly stated in the Knoxville case.

Classification of Property Units.—Simplicity of accounting requires the use of different methods for different property units. It will be plain, perhaps, that the cost of such property as fuel, oil, waste, and supplies in general, which one can see are consumed in quantity from day to day and from hour to hour, do not represent investment, as the term is generally used, but operating expense; and such property used during any fiscal period should be paid for by the public during that period. The usual fiscal period is a year, and it may be said fairly that the cost of any property used up during a year is properly a part of operating expense.

This reasoning would apply also to parts of units, such as tubes of boilers, tires on locomotive driving wheels, shingles on buildings, etc., the replacement of which, though not occurring annually, may be classed as repairs rather than as renewals. The units are not retired, they are only repaired. All replacements that properly may be called repairs may without question be provided for in operating expense.

Closely allied to fuel and other supplies, but really representing investment, are certain other classes of property, like telegraph poles, railroad ties, etc. These last for a number of years, but not all of any kind last the same number of years, and a condition gradually comes about, in a stationary or slowly growing property, that calls for a renewal of a certain fairly regular percentage of all the poles or all the ties each year. The maintenance of the normal condition of poles or ties, which condition is necessarily less than new, properly may be charged in operating expense. The depreciation actually accruing during any year is offset approximately by equivalent expenditures for renewals. Simplicity of accounting does not "require" this method of handling tie accounts, but they have nevertheless been handled in this way. The integrity of the original investment, if that is measured by the service life of physical items, is not maintained by this method, because initial depreciation is not made good.

The difference between normal condition and condition new, of such items as have been mentioned above, may be called initial decretion, producing what is sometimes estimated as depreciation. Ap-

parently, under a strict interpretation of the law, such initial depreciation must be lost to the owner in a valuation unless:

1.—He has collected an adequate allowance for depreciation from the beginning; or
2.—The whole thing is ignored as not reducing the value of the property as a going concern; or
3.—It is considered that this depreciation is a development cost which, added to the other costs, will offset itself when full depreciation is estimated.

Forming a link between such property, as ties and poles and the next distinct class to be mentioned below, is such property as the rolling stock of a great railroad. The items are much larger than the separate ties, but the aggregate yearly expenditures for renewals may be not more than those for ties, and in the past the cost of renewals has been put into operating expense as the renewals occurred, under the head of repairs and renewals; but it may be met just as well by moneys taken from a fund, maintained by sufficient annual contributions for depreciation from the patrons of the utility, or by new capital furnished by the owner in lieu of that contributed by the patrons as depreciation allowances, but invested otherwise in the property by the owner. The Interstate Commerce Commission now requires that reserves be set up for the rolling stock of railroads, but, with respect to initial depreciation existing before such reserves were established, when retirements were handled as operating expense, the owners are situated exactly as they are with respect to ties, and they must lose such depreciation in a valuation unless the commissions or Courts will accept in their favor one of the two alternatives last mentioned in the foregoing paragraph.

Theoretically, the public contributes only the cost of the items retired. The items purchased in place of these may cost more or less than the items retired. Only the cost of the items retired should be charged against the depreciation contributions, and this should be charged, whether or not a new item is purchased.

The depreciation of certain other property units, as the very large and costly structures of a great railroad, the cast-iron pipe lines, or the pumping engines of a water-works property, the buildings, pipe lines, and holders of a gas property, etc., all having long life, and some of them involving, for the maintenance of the service rendered, relatively large expenditures at irregular intervals, can be provided for best without question by regular annual allowances established for the purpose, and accruing from year to year for each item somewhat in proportion to the spent life of the item. If proper accounting is provided, it is immaterial, except as a matter of public policy and economy, whether the allowances for any particular item be held as

cash at interest or as convertible securities until that item is retired, or be invested in additions or betterments to the property. In either case the integrity of the investment is maintained.

The Replacement Method.—The replacement method is an accounting method which provides that the owner shall be reimbursed for the cost of a property unit at the time it reaches the end of its service life and is retired. That is, the cost is charged to operating expense at that time. Operating expense being collected from the public, it is said that the owner is reimbursed when the charge to operating expense is made. Some owners do not charge the cost of the retired item, but rather the cost of the item which replaces it—if such there be—which may be more or less than the cost of the item retired. If it is more, the public is contributing capital to this extent; if less, the owner is losing capital to this extent. This practice is prohibited by the Interstate Commerce Commission for large telephone companies, although permitted for many classes of railroad property.

The replacement method is the most convenient one to use for some short-lived property units, but, under a possible interpretation of the law as laid down in the Knoxville case and the Minnesota rate cases, the method is not strictly applicable to any wasting property units, because it seems not to keep the investment intact at all times. However, the method should be legally proper for railroad use for all units except equipment, because the regulating body, the Interstate Commerce Commission, has sanctioned it, and the Committee believes that a proper interpretation of the Knoxville decision would also permit the use of the method in some other cases, certainly to the extent that replacements may be properly classed as current repairs. For the language is to the effect that the public utility "is entitled to see that from earnings the value of the property invested is kept unimpaired," and that: "before coming to the question of profit at all, the company is entitled to earn a sufficient sum annually to provide not only for current repairs, but for making good the depreciation and replacing of parts of the property when they come to the end of their life."

To explain the method more fully, let railroad ties be considered: They are an elementary part of the unit known as track, and, in estimating cost, would be an item in the inventory. If an old track is properly maintained, about the same percentage of the ties will be replaced with new ones each year—ties will not all disappear and be replaced at once. In the beginning they were all new, though, in the case of a railroad of magnitude built in one operation, some of the ties first laid will have reached nearly the ends of their lives by the time the whole line is completed. What will come about with such a road will be a condition such that at any given time there will be some ties ready to be replaced, some new ties that have just been put in, and perhaps approximately equal numbers in all stages of

life, so that the whole would inventory about 50% of new condition. Ties never can be much better than this nor much worse in an old, well-maintained track. In northern latitudes they may be a little under normal in the spring, just before renewals begin and a little above normal by the end of the summer, when renewals for the season have been completed. Ties in such track are kept in normal condition with respect to investment and service if they are kept in about 50% condition by annual renewals accounted for in operating expense. This is not exactly true when there is much recently added new track, but relatively small additions do not greatly disturb the balance.

When property is maintained in this way it is said to be maintained under the replacement method of accounting; theoretically, the cost of the old item is charged off, that is, credited to capital and charged to operating expense, and the cost of the new item is charged to capital when the replacement is made, and no accumulating depreciation allowances are provided or needed. Really, no entry need be made in the capital account unless the new item is cheaper or more costly than the old, and, in the matter of ties, is not made usually in any event, ties being considered much like fuel.

It has frequently been insisted that allowances for depreciation are strictly for the purpose of making replacements, and from this point of view property units retired without being replaced would not be provided for by the replacement method, but, in modern accounting, a unit which is retired and not replaced is, and in equity should be, treated in the same way as if it were replaced. That is, its cost should be made good to the owner, and is made good by charging it to operating expense.

Before describing other methods of accounting, an effort will be made to distinguish between depreciation reserve and depreciation funds, and to state the depreciation theories on which accounting methods are based.

Depreciation Reserve vs. Depreciation Fund.—Depreciation reserve and depreciation fund are two entirely distinct and separate things which are frequently confused, largely because of the loose terminology used in the past by accountants, valuation engineers, Courts, and commissions, all of whom have used the terms interchangeably, and have sometimes increased the confusion by speaking of either as the "depreciation reserve fund." The depreciation reserve represents merely the result of a series of entries on the books, made for the purpose of preventing overstatement of earnings or property, as will be explained later; but a fund is money or its equivalent set aside to be devoted to some particular purpose, a depreciation fund being set aside to provide for depreciation of property.

On the balance sheet of any enterprise, various items of assets appear, and among these will be the value of the property. If this property is carried at its full original value or cost on the asset side,

but has been in service some time and has lost some of its value, this fact should appear on the balance sheet in some way, or the actual value of the property on any given date will be over-stated by the amount of the depreciation. Hence, in modern bookkeeping the item of depreciation reserve appears on the liability side of the balance sheet, which makes the property showing, at least so far as the effect of depreciation is concerned, correct, provided the depreciation has been properly estimated. This same result could be achieved by simply writing off a certain part of the value of the property, carrying the property on the asset side at its depreciated or written-off value and making no entry on the liability side of the balance sheet. This, however, is not considered to be as valuable a method from an historical standpoint as that which carries property at its full value on the asset side and sets up a reserve on the liability side of the balance sheet to allow for the depreciation.

When a depreciation reserve is established for the balance sheet, one or more depreciation reserve accounts are carried in the ledger. As illustrating the use of the depreciation reserve accounts, let it be supposed that a certain item of property has depreciated $100 during a given year, and that no cash has been expended for it, nor is any expenditure to be made immediately. Although no expenditure is to be made immediately, or has been made, the fact of the depreciation means the consumption of $100 of invested capital, and, unless an entry is made to show this, the net revenue for the year will be overstated. Therefore, although no cash is spent, the operating expense account is charged with $100; but, operating expense being charged with $100 which is not spent, a surplusage of cash or its equivalent to the amount of $100 will be found. To offset this and prevent the overstatement of the assets, the depreciation reserve account will be credited with $100. As time goes on, the item will continue to depreciate, and the depreciation will be charged periodically in operating expense and credited to the reserve account, until finally the item will be retired. At this time, assuming that it has no salvage value, it will be written off from the property statement on the asset side of the balance sheet, and at the same time its value must be taken from the reserve in order still to preserve proper balance. Therefore, the reserve will be charged with the whole value of the item, which, if the estimates of depreciation from time to time have been correct, will be just equivalent to the sum of the credits to depreciation reserve for this item.

Whenever a charge is made against operating expenses for depreciation, the depreciation reserve account is credited. When property is retired, the cost new less salvage recovered is charged against the depreciation reserve. At any given date, therefore, the amount that should appear in the depreciation reserve on the balance sheet would be a summary of the credit balances of the various depreciation reserve accounts appearing in the ledger. If the estimates made for

depreciation from time to time happen to agree with the fact, the amount standing in the depreciation reserve item on the balance sheet at any time will represent the accrued depreciation in the property, but this is likely seldom to be the case.

The depreciation reserve exists in modern bookkeeping, whether or not there is a depreciation fund. The depreciation fund is a part of the assets, and is established merely by charging the fund and crediting cash or some other asset item with any amount that at any time may be deemed proper. This double operation does not disturb or change the amount of the assets, but merely changes the names under which they are carried.

It should be borne in mind that the setting aside of assets in a depreciation fund is not charging depreciation. This is accomplished only by making a charge against operating expenses and so decreasing the showing of net earnings. Depreciation is not provided for until it is charged in operating expense, and it is not deducted from the property until it is credited to depreciation reserve, or written off from the property statement on the asset side.

Practically, of course, collection may be made from the public without any depreciation entry in the books, that is, enough may be collected in rates to pay operating expense, depreciation, and fair return, but that part which should be applied to the property to cover depreciation may be otherwise disposed of, as, for instance, distributed as unwarranted dividends; and, when this is done wittingly, a theft has been committed from the property, and, when done unwittingly, error of management is shown. The argument in the foregoing paragraph assumes an honest and proper management.

Under the replacement method of accounting, there would be neither depreciation reserve nor depreciation fund, the reserve being established only when operating expense is charged without expenditure of cash, and under this method at any time the property will be over-stated by the amount of the depreciation accrued but not yet realized through retirements.

If no charge is made to operating expense or credit to reserve for depreciation of an item previous to its abandonment, but specific assets are set aside as a fund to offset the accruing depreciation of the item, "depreciation fund" will appear among the assets, but, as before, the profits and the property will be over-stated at any time by the amount of depreciation accrued but not realized by retirement.

Depreciation Theories.—Depreciation of physical units used in connection with public utilities, or, indeed, with any other industries, does not proceed in accordance with any mathematical law. One unit, identical with another, and used for the same purpose in the same way, goes out of service before the other, perhaps because of some accident; or a unit goes out of service before it is worn out,

because of some new invention that displaces it, or some new requirement of service that makes its use no longer possible; another unit, performing hard service, requires so much more repair during its later than during its earlier life, and yields so many fewer units of service per unit of time, that, long before it is actually worn out, it becomes uneconomical to use it, and it is replaced some time before it is incapable of service. There is no regularity in the development of the increasing need for repairs; there is no regularity in the progress of depreciation; but, in order to devise a reasonable plan for laying aside allowances from year to year to make good the depreciation as it accrues, and to provide for the accumulation of a sum equivalent to the cost less salvage of a unit by the time it is retired, some theory of depreciation progress must be assumed on which such allowances may be based. Several such theories have been suggested, and corresponding accounting methods have been developed for some of them and have been adopted by accountants and approved by Courts and commissions. Three of the more generally mentioned theories will be explained, namely: The straight-line theory, the compound-interest theory, and the unit-cost theory; and the corresponding accounting *methods* will then be discussed.

The Straight-Line Theory.—The straight-line theory assumes that depreciation progresses uniformly from the beginning of service to the end of service life; that when service life is half gone, value less salvage is half gone, when service life is nine-tenths gone, value less salvage is nine-tenths gone. Thus, if a unit cost $100 and has a salvage value of $10, after an estimated total service life of 10 years, and has been in service for 5 years, it has depreciated in value one-half of $90, or $45, and is worth $45 plus $10 salvage, or $55; if it has been in service 9 years, it has depreciated $81 and is worth only $19. This is the simplest depreciation theory, and because of its simplicity it has been widely adopted. Its application in accounting and valuation is sometimes attended with certain apparent inequities which will appear in the discussion of accounting under this theory.

The Compound-Interest Theory.—This theory assumes that depreciation progresses at the same rate as a sinking fund grows from an annuity accumulating at compound interest. Thus, if a unit cost $100 and has a salvage value of $10 after a total service life of 10 years, an annuity may be determined such that, with its accumulation of interest, it will equal $90 at the end of 10 years, and the depreciation of the unit at the end of any year will be equivalent to the accumulation of the annuity and its accretions to that time. If the rate of interest be r; the cost less salvage of a unit be C; and the estimated service life N years; the annuity necessary to set aside to equal C in N years is given by

$$A = \frac{C\,r}{(1+r)^N - 1} \qquad (1)$$

and the sum to which this will amount in n years is given by

$$S = \frac{C[(1+r)^n - 1]}{(1+r)^N - 1} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

This may be solved by logarithms, or the result may be taken from amortization tables, many of which have been published covering Equations (1) and (2). In applying these equations to a particular example, let it be assumed as before that a service unit costs $100, has a salvage value of $10 after a total service life of 10 years; that it has served 5 years, and that interest is at the rate of 5 per cent. Then $90 must be accumulated in 10 years, and the necessary annuity, A, at 5% is $7.16. This annuity, with its accumulation of interest, will amount to $39.54 in 5 years, and the unit is then worth $50.46 plus $10.00 salvage, or $60.46. After 9 years of service, the depreciation amounts to $78.90, and the unit is worth $11.10 plus $10.00 salvage, or $21.10. The depreciation during any one year is equivalent to the annuity for that year plus the interest on the accumulation of previous annuities and interest.

This theory may have a certain practical advantage over the straight-line theory when applied to newly created properties, because it requires the setting aside of a less sum to offset accruing depreciation in the earlier years of the enterprise, when expense is likely to be relatively high and income relatively low. On the other hand, when applied to a unit for which the cost for repairs and upkeep is much greater toward the end of its life than at the beginning, this theory bunches the larger deductions from income for repairs and depreciation; and the straight-line theory would tend to produce less inequality in the deductions throughout the years of service life, so far as this influence is concerned.

With some units, such as locomotives, the expense for repairs is so large that the depreciation deduction is relatively insignificant and, so far as inequality of deductions is concerned, it is of less consequence which theory is adopted than it is when units like water pipe are under consideration, because the item of repairs on such units is relatively insignificant when compared with the accruing depreciation.

The discrepancy between accrued depreciation under the straight-line theory and under the compound-interest theory is much greater when the units are long-lived than when they are short-lived. Thus, a 10-year unit having no salvage value loses half its value in 5 years under the straight-line theory, regardless of the rate of interest, and at 5% under the compound-interest theory it loses nearly as much—about 44% of its value in the same time (45.1% at 4% and 41.6% at 7%); but a 50-year unit, losing half of its value in 25 years, under the straight-line theory, loses only 22.8% of its value at 5% under the

compound-interest theory. It will be shown later, however, that the total cost for service during the service life of a property unit, including interest on the value in the unit, depreciation annuity or allowance, and interest on both these annual costs, is exactly the same under one theory as under the other, and indeed is the same as under the replacement method of maintaining property already described, but, notwithstanding this equality, the theories are not equally applicable to all classes of property, as will also be shown later.

The Unit-Cost Theory.—This theory holds that the depreciation of a service unit at any stage of the service life is the difference between what it cost new and what could be paid for it in its depreciated condition without increasing or diminishing the cost per unit of service during its remaining service life. In other words, it is equivalent to the difference in worth of two units which would perform the same service at the same total cost per unit of service, one having an estimated service life equal to the total estimated service life of the unit under consideration, and the other an estimated service life equal to the estimated remaining service life of the unit. Comparing the relative economy of two such units in general, the engineer solves a question like this: If Unit A costs \$100 and lasts 20 years, how much can be paid with equal economy for Unit B for the same service, if it will last but 10 years? The costs must include all costs necessary to put the units in place and at work. The estimate should include consideration of the effect of the lessened output of service and greater cost for operation and repair of the units, as they draw near the ends of their lives; but this effect is sometimes ignored. When excluded, the "unit-cost theory" becomes the "compound-interest theory", which, therefore, may be considered to be a special case of the "unit-cost theory". On the basis of this theory of difference in worth, considering Unit A as the unit new and Unit B as the depreciated unit, a formula for the present worth of the depreciated unit is developed thus, ignoring output and cost of operation and repairs: The two units are compared on the basis of equal annual cost for interest and depreciation annuity. If C be the cost new of Unit A and c the present worth of the depreciated unit, B; r the rate of interest; N the estimated service life of A; and n the estimated service life of B; the total annual costs for interest and annuity will be:

$$\text{For } A \ldots \text{Interest, } C\,r, \ldots \text{Annuity, } \frac{C\,r}{(1+r)^N - 1}$$

$$\text{For } B \ldots \text{Interest, } c\,r, \ldots \text{Annuity, } \frac{c\,r}{(1+r)^n - 1}$$

Making these two equal and solving for c gives

$$c = C\,(1+r)^{N-n}\,\frac{[(1+r)^n - 1]}{[(1+r)^N - 1]}$$

and $C - c$ would be the depreciation during $N - n$ years, which will be found to be identical with the S of Equation (2), if $(N - n)$ be substituted for n in that equation. This is the compound-interest theory.

If the items of increased cost of operation and repairs be included, the application of the unit-cost theory becomes quite complex. The comparison is made in the same way by making total annual costs equivalent, but the annual cost is first reduced to cost per unit of service by dividing by the estimated output capacity of the new and depreciated property units. Thus, if A and a are costs of operation and repairs for the longer- and shorter-lived units (new and depreciated units), respectively; C and c the respective costs, C being the actual or estimated cost of the new unit of longer life and c the price that could be paid for the shorter-lived (depreciated) unit to produce the same cost per unit of service; B and b the respective depreciation annuity multipliers of C and c under the sinking-fund theory; if D units of service may be realized annually from the longer-lived (new) unit and d units from the shorter-lived (depreciated) unit; and if r be the rate of interest, the following equations of annual costs may be written:

$$\frac{A + B\,C + r\,C}{D} = \frac{a + b\,c + r\,c}{d}$$

Solving this equation for c, the price that could be paid for the shorter-lived (depreciated) unit is found to be:

$$c = \frac{\frac{d}{D}(A + B\,C + r\,C) - a}{b + r}$$

The difference between C and c is the accrued depreciation at the date of estimate. The amounts of B and b may be obtained from

$$B = \frac{r}{(1 + r)^N - 1}$$

and

$$b = \frac{r}{(1 + r)^n - 1}$$

or from amortization tables. The capacities, D and d, of the new and old units should be their actual or estimated capacities for service under normal conditions, not the respective outputs under varying business conditions which might readily make the older unit more valuable than the newer one.

The compound-interest theory produces, in its application in accounting, equal annual charges for the sum of interest and depreciation allowances, while the unit-cost theory is based upon equal

annual charges for the sum of interest, depreciation allowance, and operating expense (including repairs) *per unit of output.* This theory is probably the soundest theory of depreciation, and, when applied with intelligence, probably furnishes the truest measure of accrued depreciation. It is not readily applicable to accounting purposes, because its formulas cannot be tabulated, but it may be used for the purpose of checking the results of the application of other theories and for finding the accrued depreciation of any property at any time, in connection with a public valuation, when there has been no proper accounting in the past, or for finding remaining value for the purpose of inaugurating a proper accounting system for an old property.

Methods of Accounting Applicable to the Straight-Line and Compound-Interest Theories of Depreciation.

This discussion will be approached from the standpoint of the proper payments of the public to company owners and the proper disposition of these payments by the company owners, rather than from the standpoint of the bookkeeping methods of the company owners. It should be remembered here that it is the duty of the public served to pay operating expense incurred in its service, interest on capital invested for its service, and the cost of property consumed in its service; but operating expense is omitted from this discussion as being independent of the other two items, which are to some extent mutually dependent on each other, and because it is only depreciation accounting that is now under consideration.

Accounting Under the Straight-Line Theory.—Under this theory the public will pay the full amount of the depreciation as it accrues, a certain uniform fraction of the whole cost, less salvage value, of any unit each year of its life, and, paying this each year, thus returning a part of the invested capital, it should pay a correspondingly decreased interest on invested capital each succeeding year. A tabular statement of a particular example will illustrate: A certain unit costs $100, has a life of 20 years, and no salvage value. The cost or book value in the unit and the payments of the public with interest at 5% are shown in Table 1.

Remembering that the property must be considered to be a continuing property and that the company owner is bound to preserve the integrity of the investment, it must be clear that the allowances paid by the rate-payers belong to the property and must be considered to be a part of it. If taken out of the property, the investment is impaired, and if paid to the stockholders, then, although they lose nothing, the bondholders' security is reduced, and the public and later stock purchasers are not protected against the gradual destruction

of the property that should be a continuing property. Practically, therefore, the depreciation allowances paid by the rate-payers must be considered a part of the property, not to be distributed to the stockholders, but to be kept in the property in some form, in order to preserve the integrity of the investment.

TABLE 1.—ILLUSTRATION OF THE STRAIGHT-LINE METHOD.

Showing Book Value and Payments by the Public on Account of Fair Return and Depreciation.

Property Unit Costing $100.00, Lasting 20 Years, No Salvage Value, Interest at 5 Per Cent.

(1) Age.	(2) Remaining value at end of year.	(3) Depreciation allowance.	(4) Return on remaining value, 5%.	(5) Combined depreciation and return, 5%.
0	$100.00			
		$5.00	$5.00	$10.00
1	95.00			
		5.00	4.75	9.75
2	90.00			
		5.00	4.50	9.50
3	85.00			
		5.00	4.25	9.25
4	80.00			
		5.00	4.00	9.00
5	75.00			
		5.00	3.75	8.75
6	70.00			
		5.00	3.50	8.50
7	65.00			
		5.00	3.25	8 25
8	60.00			
		5.00	3.00	8.00
9	55.00			
		5.00	2.75	7.75
10	50.00			
		5.00	2.50	7.50
11	45.00			
		5.00	2.25	7.25
12	40.00			
		5.00	2.00	7.00
13	35.00			
		5.00	1.75	6.75
14	30.00			
		5.00	1.50	6.50
15	25.00			
		5.00	1.25	6.25
16	20.00			
		5.00	1.00	6.00
17	15.00			
		5.00	0.75	5.75
18	10.00			
		5.00	0.50	5.50
19	5.00			
		5.00	0.25	5.25
20	0			
Totals		$100.00	$52.50	$152.50

The allowances, being part of the property, ought to earn at the same rate as the capital remaining in the property. That is, persons having invested in a given property do not wish the investment continually to vary either in amount or return rate, they desire that the investment shall remain and shall earn at a uniform rate. The allowances may be made to earn at the same rate as the remaining property, if reinvested immediately in the property; they may earn at an even greater rate, if fortunately invested otherwise; or they may and probably will earn at a less rate, if placed in bank. Immediate investment in the property itself, which should insure a continuance of the full rate of return, may not be possible, often will not be when a property is not rapidly growing, and lack of opportunity for other investment may require the banking of allowances. It would seem, therefore, that the public should make up the shortage in return, if any there be. Theoretically then, proper accounting, under regulation that determines fair return by which full return on the entire investment is secured, would charge to operating expense any deficiency of return on allowances and credit to income any excess of return over the fair rate.

When only a single item is considered, this method, as the table shows, makes payments from year to year ununiform, with much larger payments in the beginning than toward the end, which is undesirable unless repairs are enough greater toward the end to equalize the payments. When there are many items of varying ages and expectations of life, this defect largely vanishes. This is not a defect of the accounting, but a necessary result of using the straight-line theory of depreciation.

Accounting Under the Compound-Interest Theory.—Under this theory, the public pays interest on the invested capital and depreciation as it accrues, according to the compound-interest theory, but there are different ways of considering these payments and distributing them in the accounting, leading to two or more different methods of accounting. The Committee will discuss two methods, namely: 1, the compound-interest method, heretofore called the equal-annual-payment method, and 2, the sinking-fund method.

1. *The Compound-Interest Method.*—Confining the discussion to a single unit without salvage value, for simplicity, the amount of the annual payment for interest and depreciation is found thus: If C be the cost of the unit; N the years of service life; r the rate of interest for fair return; and r' the rate to be used in amortizing the property; the total annual payment, P, by the public for fair return and depreciation is given by

$$P = Cr + \frac{Cr'}{(1+r')^N - 1}$$

the second term being evaluated by logarithms or taken from amortization tables. It will perhaps be plain that this should be the payment for the first year, but for the second year the payment should include interest on the amortization annuity of the first year, if the public served is to pay the full depreciation as it accrues, and the return should be on the original cost reduced by the first annuity, just as in the straight-line method. For the third year, the cost, or value, should be reduced by the depreciation payments of the first two years, and the depreciation payment should include interest on the sum of the two preceding depreciation payments, and so on. Now it may be shown that if r' equals r, all these several annual total payments for interest and depreciation are identical in amount and equal to that first above written for the first year. If r' does not equal r, the total payments are not the same throughout the years. Table 2 will show the remaining value, depreciation accruing, and total payments on account of fair return and depreciation, when the amortization interest rate is assumed the same as the fair rate of return—5%, and when the latter is the larger—7 per cent.

This method of disposing of the payments of the public on account of fair return and depreciation was called, in the Progress Report, the equal-annual-payment method, because it seemed to make the total payments on account of operation, return, and depreciation more nearly equal from year to year than other methods that had been discussed. As it does not necessarily do this in all cases, it might better be distinguished from other methods by a more exact title. Under this method the total payment of the public served, for interest and depreciation, is divided into two parts, one the full annually accruing depreciation under the compound-interest theory, and the other, return on the depreciated cost of the unit. This is exactly analogous to the straight-line accounting method, the only difference being in the amount of the accruing depreciation. It is the complete and logical method of accounting for the payments of the public for interest and depreciation under the compound-interest theory of depreciation, and the method will be given the name of the theory and will be called the "compound-interest method", logically comparable with the use of the "straight-line method" under the "straight-line theory". Although the name "equal-annual-payment" has gotten into valuation and accounting literature, this literature is in a transition stage, and the Committee thinks there is good reason in the preceding argument for the new name.

2. *The Sinking-Fund Method.*—Sinking fund is a very old term and has long been connected with the plan of providing for the payment of a debt, which is to become due in the future, by setting aside and investing a properly determined annuity, such that, with accumula-

TABLE 2.—Illustration of Compound-Interest (Equal-Annual-Payment) Method of Accounting for Depreciation.

Assumptions: Property having 20-year Life, Valued when New at $100. Computations of Depreciation Allowances Based on 5% Interest, Compounded Annually. Annual Return on Capital Invested, 5 and 7 per cent.

Age, in years.	Remaining value at end of year.	Depreciation during year = depreciation allowance.	Return on Remaining Value of Property at:		Combined Depreciation and Return	
			5%	7%	5%	7%
(1)	(2)	(3)	(4)	(5)	(6)	(7)
0	$100.00					
		$3.02	$5.00	$7.00	$8.02	$10.02
1	96.98	3.18	4.84	6.79	8.02	9.97
2	93.80	3.33	4.69	6.57	8.02	9.90
3	90.47	3.50	4.52	6.33	8.02	9.83
4	86.97	3.68	4.34	6.08	8.02	9.76
5	83.29	3.86	4.16	5.83	8.02	9.69
6	79.43	4.05	3.97	5.56	8.02	9.61
7	75.38	4.26	3.76	5.27	8.02	9.53
8	71.12	4.47	3.55	4.98	8.02	9.45
9	66.65	4.69	3.33	4.67	8.02	9.36
10	61.96	4.93	3.09	4.33	8.02	9.26
11	57.03	5.17	2.85	3.99	8.02	9.16
12	51.86	5.43	2.59	3.63	8.02	9.06
13	46.43	5.70	2.32	3.25	8.02	8.95
14	40.73	5.99	2.03	2.85	8.02	8.84
15	34.74	6.29	1.73	2.43	8.02	8.72
16	28.45	6.60	1.42	1.99	8.02	8.59
17	21.85	6.93	1.09	1.53	8.02	8.46
18	14.92	7.28	0.74	1.04	8.02	8.32
19	7.64	7.64	0.38	0.53	8.02	8.17
20	0.00					
		$100.00				

tions of interest, the amount of the funds in hand when the debt matures, will be equal to the amount of the debt. Only in comparatively recent years has the method been used for the amortization of wasting properties. It is not so well adapted to this purpose, as to

the amortization of a debt, particularly when the property is a public utility. The method contemplates the creation and maintenance of a fund which, considering the danger that it may be misapplied, is against good public policy. Moreover, when a property is growing it is good economy to use the annuities in extensions of the property; but there is danger, when this is done, that confusion in the accounting will result, and there is always danger that some persons may confuse annuities without their accretions, with the depreciation that the annuities plus their accumulations of interest are intended to cover. Under this method, the public served is credited with paying only the annuity toward depreciation, its payments on this account, therefore, being equal from year to year. The remainder of the total payment for interest and depreciation is treated as fair return computed on the undepreciated cost of the unit concerned. As this is also always the same, the total payments of the public for fair return and depreciation are always the same throughout the years, and the amount is that above written,

$$P = C\,r + \frac{C\,r'}{(1+r')^N - 1}$$

If r' equals r, the total annual payments agree in amount with those of the compound-interest method, but, if the return rate is higher than the amortization rate, as has been usual in practice heretofore, the total annual payments by the sinking-fund method are larger in the later years than those of the compound-interest method as shown in Column (7) of Table 2, and thus the company owner is overpaid. It has already been argued that the same rate of return should be earned by all capital kept in or held for the property, and hence it should be true that amortization rates of interest should be the same as fair return rates, and they will be so considered in the remaining discussion. Thus, the total annual payments by both methods will be identical. As full payment for interest and depreciation is made under the compound-interest method and the same totals are paid under the sinking-fund method, full payment must be made under this method.

To defend crediting the public with only the annuities toward depreciation and full return on the undiminished cost of service units, one of two arguments is invoked:

1.—The annuities are paid to the company before it has occasion to use them to retire the units they are intended to cover; hence, the company owner can earn interest on them, and therefore they are discounted and considered parts of a fund held in trust by the company owner and invested for the purpose of earning that interest necessary to make it amount to the payment required of the public when the unit covered is retired. As the company may not earn distributable profits

on these annuities, it must be allowed its fair return on the undiminished cost of the units covered until such time as the fund matures, which in theory should be when the item is retired. If the fund is then immediately reinvested in the property, the cost value goes on as before; if, for any reason, it is withdrawn from the property, the cost value should be reduced accordingly.

2.—The public is assumed to make full payment on account of depreciation yearly as it accrues; but, because it makes these payments in advance of the time when they must be used to retire the unit they cover, the company owner will be able to use them to earn interest, and hence the public charges the company for the use of these payments assumed to be due only when the item is retired; that is, the public pays the present worth of payments assumed to be due on the retirement of the unit covered. But, again, the company owner must invest these annuities so that with their earnings they shall equal the cost of the unit covered when it is retired; they cannot be used to earn distributable profits, and, therefore, as before, the public should pay a fair return on the undiminished cost value of the units concerned until they are retired or the payments mature. Under this conception, it must be assumed, either that the last payment due is made first and discounted, or that the growth of depreciation is reversed, the larger annual growth being at the beginning and the smaller at the end of service life. The conception seems not to be defensible.

Comparisons.—The entries to be made in the accounting under the compound-interest and sinking-fund methods must of necessity be different. Under the compound-interest method, the book value of the service unit must be reduced from year to year in the amount of the full accruing depreciation by a credit to depreciation reserve; under the sinking-fund method, the public paying return on the undiminished cost, there should be no deduction from book value until the unit is retired or the sinking fund matures, which times are theoretically the same. Thus, the sinking-fund method becomes, so far as the property account is concerned, the same as the replacement method of maintaining property. Unfortunately, this has not been understood clearly, and the sinking-fund method of accounting has not proceeded under this theory; confusion has resulted, and, indeed, is likely to result, from any attempt to account for depreciation under this method.

The Entries Under the Compound-Interest Method are, and are explained, as follows: Income in the amount of the depreciation allowance is charged to operating expense to keep the income statement correct. Depreciation reserve is credited in the same amount to keep the property statement correct. We now have assets consisting of property depreciated by the depreciation reserve and cash or other assets equivalent to that allowance. The depreciated property will earn

fair return the next year, and the allowance also should earn fair return. It may be—and should be, if possible—at once re-invested in the property, but, however it is used, unless retained as working capital, cash will be credited and the particular asset purchased will be charged, thus making no change in the total assets. If the allowance is invested in the property, fair return presumably will be earned. If this is not immediately possible, so that the allowance must be held for a time until needed in the property, it should be invested in outside securities, and the deficiency of fair return, if any, should be charged next year in operating expense, and the excess above fair return, if any, should be credited to income, reducing the necessary payment of the public on account of depreciation. Only on such an assumption is it fair to use fair rate of return in computing annuities. If the company owner must take his chances of earning more or less than the fair rate of return on the annuities, then that rate should be used which experience shows to be probably obtainable. In the case of a reasonably growing property, investment in the property will usually be possible though perhaps not immediate; in the case of a stationary property, it will in general be impossible.

There is no ambiguity in the foregoing described method; no chance for confusion; the method is exactly analogous to that described under the straight-line theory of depreciation.

The Entries Under the Sinking-Fund Method should be, and are explained, as follows:

Operating income, in the amount of the annuity, is charged to operating expense in order to keep the operating income account correct. As fair return is based on book value, there must be no reduction in this below original value, except as units are retired, so there will be no credit to depreciation reserve, which will reduce book value, but to some other liability account as trust fund reserve. Among the assets, cash or other assets will be credited, and sinking fund charged with the amount of the depreciation annuity. The fund will be credited only as cash is expended for the replacement of, or in lieu of, units covered by the fund. The fund must be carried as an undistributable asset belonging to and going with the property, to offset the trust fund reserve. Accounting must go on within the fund; its cash must be invested and its income accounted for separately from the operating income. If its cash is reinvested in the property itself, purchasing new units and increasing the income producing but depreciating property, the earnings from such new units should be charged to the fund. These units being depreciating units must also earn depreciation annuities, which must in turn be invested and in effect a new sinking fund must be set up for each addition to depreciating property. It would be manifestly impossible to separate the earnings of the several units of a complex property, but the operating income

can be separated into two parts, respectively proportioned to the investment from the fund and that from other capital, and its due share charged to the fund. This will be but an approximation to the truth and again it must be remembered that the additions are of depreciating property. Moreover, in a valuation it will be impossible to distinguish between units purchased from the fund and those acquired otherwise. If the fund is invested outside the property, no difficulty arises in determining the income. Theoretically, if it is invested so that it earns less than the rate assumed in determining the annuities (which, as has been pointed out, should be the fair rate of return), the deficiency may be put in operating expense and so collected from the public; and, if it earns more than the fair rate of return, the excess should be credited to the public sinking-fund reserve to the relief of the public. However the fund is invested, it must be held to be a part of the undistributable property, an asset to be included in a valuation offsetting trust fund reserve.

The foregoing procedure is more complex than that involved in the compound-interest method, and the sinking-fund method has no advantage over that method in any way. It does not show directly in the property account what the accrued depreciation is at any time, as does the compound-interest method, but presents the fiction—possibly justified by the existence of the trust fund reserve—of an undepreciated property. It must be remembered that the company owner has two elements of value in his property—money and time—that is to say, cost and length of service life, both of which must be protected. It is not enough that when a property unit goes out of service there shall be a new unit of equal cost to replace it, but the new unit must have a service life equal to that of the retired unit, or, there must be more units of less life, or less units of more life. Under a strict application of the sinking-fund method, when the fund is invested in the property, the foregoing result is realized only if the funds be set up for each separate addition, purchased with fund money, based upon the cost and service life of the addition. The intricacies of this method when strictly followed, the confusion likely to result when the fund is invested in additions to the property, and the desirability of such (latter) form of investment, seem to make this method of accounting, for depreciation of a growing property, undesirable.

The sinking-fund methods in common use in the past have varied from the foregoing procedure. Either annuity or full amount of accruing depreciation has been charged to operating expense, and credited to reserve, thus reducing book value. When only the annuity has been charged and credited, the property account shows depreciation, but not enough, and explanation must be forthcoming to justify the accounts and the distribution of public payments. The accounts

do not tell the whole story, as in the compound-interest method. If the full accruing depreciation is charged and credited, no sinking fund is required, and no explanation can justify any other distribution of the public payments than that of the compound-interest method which this procedure then becomes.

The sinking-fund methods heretofore generally used are incomplete accounting methods, under the compound-interest theory, in that outside explanation is required to justify the accounts and the distribution of public payments for return and depreciation. Since, as has been said before, there is full and complete payment by the public for return and depreciation under the compound-interest method, and since, with equal rates for return and annuity, the payments are the same in total amount under the sinking-fund method, they must be complete under both methods, and it would appear that the sinking-fund method so widely used must have been the outgrowth of a failure to comprehend the problem completely. At any rate, it seems no longer desirable.

Comparison of Four Methods.—

(*a*)—Compared to Bond Payments.—The four methods of accounting for depreciation may be compared to four ways of making and paying bonds. Thus, the replacement method corresponds to a bond payable n years from date, with interest at the rate of r per cent. per annum until paid; the straight-line method corresponds to a set of n serial bonds of equal amounts, with interest at the rate of r per cent. per annum, giving diminishing annual payments; the compound-interest method corresponds to a set of n serial bonds of unequal amounts, respectively equivalent to the annuity accumulations of the several years that the bonds run, giving equal annual payments. The sinking-fund method corresponds to bonds payable n years after date, and requiring a trust fund to be accumulated with a bank or trust company to meet the payments when due, also giving equal annual payments.

(*b*)—On a Basis of Costs.—To compare the four methods thus far described on a basis of cost, it will be assumed that the property is made up of a single unit, costing $100, having no salvage value, and lasting 20 years—of course, a purely hypothetical case. If interest for all purposes is at 5%, the annual and total payments by the public to the owners will be as in Table 3.

In preparing Table 3, the return under the replacement and sinking-fund methods has been figured as 5% on the undiminished cost of the property unit, while under the straight-line and compound-interest methods it has been based on the depreciated value of the unit.

The headings in Columns 9 and 10, while representing what is frequently considered to be "depreciation" and "return", are not expressive of the facts. The figures in Column 9 represent the sinking-fund annuity for each year, which is only a part of the depreciation, as may

be seen by reference to the total at the foot of the column, which is only $60.40 instead of $100. The remainder of the depreciation is incorporated in the figures in the column headed "Return", where in each year it represents 5% on the difference between the undiminished cost of the unit and its depreciated value in that year. Of the $100 represented by the total of this column, $39.60 is "depreciation" and $60.40 is "return".

TABLE 3.—Comparison of Combined Annual Depreciation and Return Upon a Property Unit of 20-Year Life, Under Different Methods of Providing for Depreciation. Assumed Rate 5 Per Cent.

Year (end of).	Replacement Method.	Straight Line Method.			Compound-Interest Method.			Sinking-Fund Method.		
	Return.	Depreciation.	Return.	Total.	Depreciation.	Return.	Total.	Depreciation.	Return.	Total.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
1	$5.00	$5.00	$5.00	$10.00	$3.02	$5.00	$8.02	$3.02	$5.00	$8.02
2	5.00	5.00	4.75	9.75	3.18	4.84	8.02	3.02	5.00	8.02
3	5.00	5.00	4.50	9.50	3.33	4.69	8.02	3.02	5.00	8.02
4	5.00	5.00	4.25	9.25	3.50	4.52	8.02	3.02	5.00	8.02
5	5.00	5.00	4.00	9.00	3.68	4.34	8.02	3.02	5.00	8.02
6	5.00	5.00	3.75	8.75	3.86	4.16	8.02	3.02	5.00	8.02
7	5.00	5.00	3.50	8.50	4.05	3.97	8.02	3.02	5.00	8.02
8	5.00	5.00	3.25	8.25	4.26	3.76	8.02	3.02	5.00	8.02
9	5.00	5.00	3.00	8.00	4.47	3.55	8.02	3.02	5.00	8.02
10	5.00	5.00	2.75	7.75	4.69	3.33	8.02	3.02	5.00	8.02
11	5.00	5.00	2.50	7.50	4.93	3.09	8.02	3.02	5.00	8.02
12	5.00	5.00	2.25	7.25	5.17	2.85	8.02	3.02	5.00	8.02
13	5.00	5.00	2.00	7.00	5.43	2.59	8.02	3.02	5.00	8.02
14	5.00	5.00	1.75	6.75	5.70	2.32	8.02	3.02	5.00	8.02
15	5.00	5.00	1.50	6.50	5.99	2.03	8.02	3.02	5.00	8.02
16	5.00	5.00	1.25	6.25	6.29	1.73	8.02	3.02	5.00	8.02
17	5.00	5.00	1.00	6.00	6.60	1.42	8.02	3.02	5.00	8.02
18	5.00	5.00	0.75	5.75	6.93	1.09	8.02	3.02	5.00	8.02
19	5.00	5.00	0.50	5.50	7.28	0.74	8.02	3.02	5.00	8.02
20	5.00	5.00	0.25	5.25	7.64	0.38	8.02	3.02	5.00	8.02
Depreciation	100.00									
Total.	$200.00	$100.00	$52.50	$152.50	$100.00	$60.40	$160.40	$60.40	$100.00	$160.40

Although the total payments by the public for return and depreciation appear from the footings, under the various headings, to be different under the different theories, this apparent variation is the result of ignoring interest on the payments or, in other words, the effect of time.

The real total cost of the unit in the 20 years, for interest and depreciation, is the accumulation of the annual allowances for these items at compound interest during the 20 years, and this is the same for the four methods and is equivalent to the sum to which

the cost of the unit would amount at compound interest in the life time of the unit, as will be shown.

Referring to Table 3: by the replacement method, the total return of $5.00 each year may be considered as an annuity which at 5% in 20 years amounts to $165.33, to which must be added $100.00, the retiring payment, making $265.33. By the straight-line method, the payment for the first year is $10.00, which at compound interest for 19 years amounts to $14.57; the second payment is $9.75, which in 18 years amounts to $13.92; and so on, the total for the 20 years summing $265.33, as before. The payments by the two other methods are alike, $8.02, and these, treated as annuities, also amount to $265.33 in 20 years; but $100.00, the first cost of the unit, if placed at compound interest at 5% for 20 years, also amounts to $265.33. Thus is the statement of equal cost for the four methods and its equivalent shown for the particular case. It may be proved mathematically, as follows:

The sum to which any principal, P, will amount at r compound interest in n years is

$$S = P\,(1 + r)^n \ldots\ldots\ldots\

for interest, as the depreciation is deducted from the principal. The series of payments for interest may be expressed as follows:

First year..... $P\,r$

Second year..... $\left(P - \frac{P}{n}\right) r$

Third year..... $\left(P - \frac{2\,P}{n}\right) r$

etc.,

nth year..... $\left(P - \frac{(n-1)\,P}{n}\right) r$

The total annual contribution then will be

First year..... $\frac{P}{n} + P\,r$

Second year..... $\frac{P}{n} + \left(P - \frac{P}{n}\right) r$

Third year..... $\frac{P}{n} + \left(P - \frac{2\,P}{n}\right) r$

etc.,

nth year..... $\frac{P}{n} + \left(P - \frac{(n-1)\,P}{n}\right) r$

Substituting those separate sums in a series of equations based on Equation (3) making the exponent of the parenthesis $n-1$ for the first equation, $n-2$ for the second, and $n-n=0$ for the last equation, there results:

For the first-year allowance.. $S = \left(\frac{P}{n} + Pr\right)(1+r)^{n-1}$

For the second-year allowance. $S = \left[\frac{P}{n} + \left(P - \frac{P}{n}\right) r\right](1+r)^{n-2}$

For the third-year allowance. $S = \left[\frac{P}{n} + \left(P - \frac{2\,P}{n}\right) r\right](1+r)^{n-3}$

etc.,

For the nth-year allowance..... $S = \frac{P}{n} + \left(P - \frac{(n-1)\,P}{n}\right) r$

and the sum of these several sums is $P\,(1+r)^n$, the same as given by Equation (3).

By the sinking-fund method, the annuity payment, Equation (4), is $\frac{Pr}{(1+r)^n - 1}$, and the total annual payment is this plus interest, Pr.

Hence the total cost in n years is the accumulation of these two items or, substituting, $\frac{Pr}{(1+r)^n - 1} + Pr$ for the F of Equation (5),

$$\text{Total cost} = Pr \frac{\left[\frac{1}{(1+r)^n - 1} + 1\right]\left[(1+r)^n - 1\right]}{r} = P(1+r)^n \ldots (7)$$

or the same as by the replacement method, and as Equation (3).

Since the compound-interest method involves exactly the same annual contributions as the sinking-fund method, the total cost must be the same, or, $P(1+r)^n$.

For the particular example assumed, namely, a $100.00 unit lasting 20 years, with interest at 5%, the total sacrifice for the 20 years, by any one of the methods, is

$$S = P(1+r)^n = \$100.00\ (1.05)^{20} = \$265.33.$$

(*c*)—With Respect to Legality and Safety.—The four methods showing identical costs, any one of them seeming to be most convenient may be chosen for a particular property or for a part of a property, provided only that it is legal, safe and fair, but the sinking-fund method, being incomplete within itself, is not recommended. The replacement method is open to the danger that a Court, valuing a property at any time, will charge against it whatever depreciation may be found, holding that depreciation of physical property is always loss of value of investment, and that, under the Knoxville decision, it must be deducted, whether or not, under accounting methods theretofore used, the depreciation had been collected from the public. Thus, the property would lose the accrued obligation of the public to pay for depreciation, which obligation must be considered an asset of the property in an equitable treatment of property maintained under the replacement method of accounting. When the method is ordered or authorized by a public regulating body, as has been done in the past by the Interstate Commerce Commission with respect to all railroad property, and is still approved for most classes of such property, it should be considered to be legal and should be safe, but not always has it been safe.

The straight-line and compound-interest methods plainly are safe, as they return the full amount of the depreciation presumably as it accrues.

The sinking-fund method perhaps requires interpretation to make clear when it is safe, as the public apparently does not return the full amount of the depreciation to the owner, and unless fully understood and properly applied by regulating or valuing bodies, it is not safe. Since the total annual payments for interest and depreciation by this method are exactly the same as by the compound-interest method, it must follow that if full payment is made in one case it is also made in the other, and that it is merely the bookkeeping distribution of these

payments which makes it appear that full payment is made in one case and not in the other. Under the sinking-fund method of accounting, the annuity assumed to be paid by the public, together with the interest on the accumulated allowances, must be charged each year to the fund held in trust for the property. The fund must be treated as a trust fund belonging to the property, or due from the public, and equivalent to physical property of equal value.

If for the purpose of public purchase or capitalization a valuation is made at any time of a property maintained under this method, depreciation will exist in amount equivalent to the accumulation of depreciation annuities and interest, if the various original estimates on which the annuities were figured have agreed with the facts as they have developed in the life of the property. To the depreciated value found will be added the accumulated allowances not invested in the property but treated as an asset due from the public and neither the company owner nor the public will lose anything, since 100% of original value will be found and the accounting may go on as before. If part of the fund is invested in additions and betterments, it will be difficult, if not impossible, to separate these from the original property, and they will be included ordinarily in the valuation; but the fund accounts should show what sums have been thus invested and what amounts remain in the fund as other assets. Only the portion of the fund not invested in the property can be added to the depreciated value. Again 100% of original value will be found to result, and purchase price will be so fixed, or the accounting may proceed as theretofore. The sinking-fund method, being likely to cause confusion and possible injustice, should not be used in depreciation accounting, but, if used, difficulties may be somewhat avoided if the fund is kept invested in assets wholly distinct from the property.

If a valuation is for the determination of fair return for the future, either of two courses is open: (1) The property may be listed at full value for the determination of interest, and full value and total service life for the determination of depreciation annuity; or, (2) The property may be listed at depreciated value for the determination of interest, and depreciated value and remaining service life for depreciation annuity. Either method is equitable, and both give the same total annual allowance. Thus, referring to Table 2, if a 20-year item costing $100.00 new has been in service 15 years, and interest is figured at 5%, the theoretical remaining value, under the compound-interest theory, is $34.74. The interest for the succeeding year is $1.73, and the depreciation annuity to retire the item in 5 years is $6.29, or a total of $8.02, which is just what is required to cover interest and annuity for the item considered new. A danger here—shown by experience to be real—is that the property will be listed at depreciated value for interest and full value and total service life for depreciation annuity.

Rate on Depreciation Allowances.—As having some bearing on accounting methods, the disposition of allowances, and the rate to be used for these allowances, the Committee desires to emphasize again the fact that the entire reasonable sum devoted to public service is entitled to a uniform fair return. While thus devoted, and while, as a matter of convenience or to conform to law, allowances for depreciation may be returned to the company owner approximately as the depreciation goes on, thus theoretically reducing the capital in the service, yet the obligation of the owner to keep the investment in the property intact, makes it improper for him to consider these allowances other than as a part of the property not to be distributed as income, or returned to his pocket, without corresponding reduction of capital, if not needed for the maintenance of the property. Therefore, he is entitled to a full fair return on the undiminished investment so long as its value is properly maintained in accordance with law. Hence the allowances returned to the investor to cover accruing depreciations should earn the same fair return as the value remaining in the property. To accomplish this these allowances are best invested in the property, but, when this is not possible, the amounts not thus invested should be placed in a fund sacred to the property, and the earnings of this fund should be accounted for—if less than a fair rate of return, the deficiency should be charged in operating expense; if more, the excess should be credited to income. Thus only will the public pay its just payment and the property receive its fair return and no more. This may be difficult or impracticable of exact accomplishment.

This means that interest on depreciation allowances should be at the same rate as the fair rate of return, and that this is the rate to use in computing annuities, and allowances under the compound-interest theory.

Accounting Depreciation, the Result.—As a result of approved accounting methods that include a reserve account and that have been used from the beginning, there will appear on the books at any given time a sum that may be called accounting depreciation. For those items maintained by the replacement method and for which reserves are not set up the sum will be zero, there is no accounting depreciation under this method. The Interstate Commerce Commission has recommended that reserves be established for all properties, and there is much to be said in favor of this recommendation. If a reserve account is set up, the balance in that account shows the amount of accounting depreciation for property in use.

If no reserve account is set up, there is no account that will show at once the amount of the accounting depreciation. The way to determine it is to compare the presently carried property statement with the original costs of the property in use, if these can be found;

the difference is the accounting depreciation for the methods of bookkeeping in use. Under the sinking-fund method, there should be no reserve set up against the property, but the balance in the trust fund reserve, which should in this case equal the fund, is the accounting depreciation.

Unless reserves are set up for all property, as has been recommended by the Interstate Commerce Commission, or a full depreciation fund is maintained, accounting depreciation can never agree with the depreciation present at any given time, but it should agree substantially with it for those units or items which are maintained from the beginning under depreciation allowance methods, if the estimates of service life on which the allowances have been based have been correct, and the property has worn at the rate assumed by the particular allowance method adopted.

The straight-line method will show at any time a larger accounting depreciation than either of the compound-interest methods.

Summary.—None of the allowance methods works out in practice exactly as indicated in the tables. Accidents, and errors of judgment in prophetic estimates of service life, may cause sums received for depreciation to be excessive or deficient, but, in the accounting of a continuing property, these differences can be adjusted from time to time.

In the opinion of the Committee, the several methods described are respectively more particularly applicable as follows:

The replacement method is applicable to those short-lived properties or parts of properties made up of a large number of items, the replacement or retirement of which proceeds after a time with fair regularity and causes no troublesome variations in return or service rates. Railroad ties are an example. Rails, when a part of a very large property, and machines where very many are owned and used, may also be handled by the replacement method. Although applicable to all such properties or parts of properties, the replacement method is at present undesirable because it may result in loss to the company owner in the case of a valuation, determined under an interpretation of the Knoxville decision, which holds that all loss of service life of physical items, except such as is made good by current repairs, is loss of value of investment, and that obligation of the public to pay can never be accepted as an asset to offset accrued depreciation.

The straight-line method of accounting applies to any property units having more than a year of service life, which are assumed to depreciate according to the straight-line theory. The compound-interest theory applies similarly to property units assumed to depreciate according to the compound-interest theory. Under either method it may be necessary to maintain a fund not invested in the property itself, as when the property is stationary or consists of only a very few large

units of long life. For such properties the sinking-fund method of accounting could be adopted, if the compound-interest theory is held to apply, provided it is fully understood and correctly applied, but it is not recommended.

The great discrepancy in the growth of depreciation of long-lived units under the straight-line and compound-interest theories should be carefully noted when determining which theory to use. For short-lived items there is not much difference, and in some cases the straight-line theory may result in more uniform annual payments than the compound-interest theory; but, as already pointed out, there is a very great difference in the results of the two theories when applied to long-lived units. For such units, the Committee believes that the compound-interest theory is more nearly in accord with the truth as expressed by the unit-cost theory, which is thought to give results most closely corresponding to actual depreciation, and here the compound-interest theory would seem best to apply.

The extent of the difference in the amount of depreciation under the straight-line and compound-interest theories, for property units having various lengths of service life, is shown in Table 4. It is to be noted that the first column shows the percentage of the total service life which has expired, and consequently the percentage of existing depreciation computed under the straight-line theory. The remaining columns show the corresponding percentage of depreciation under the compound-interest theory based on a 5% interest rate compounded annually. For example, when 20% of the life of a property unit has expired, the depreciation by the straight-line theory would be 20%, regardless of the length of life of the unit, but by the compound-interest theory the depreciation would range from 16.3% for a unit having a life of 10 years to 1.3% for a unit having a life of 100 years.

TABLE 4.

Percentage of total life which has expired = percentage of depreciation by straight-line theory.	Percentage of Depreciation Under Compound-Interest Theory, Corresponding to Percentage of Loss of Service Life Given in First Column.				
	10-year unit.	25-year unit.	50-year unit.	75-year unit.	100-year unit.
10	8.0	5.5	2.6	1.2	0.5
20	16.3	11.6	6.0	2.9	1.3
30	25.1	18.6	10.3	5.3	2.5
50	42.9	35.3	22.8	13.8	8.0
70	64.7	56.6	43.1	31.6	22.5
90	87.7	83.8	76.3	68.6	61.1

This report is particularly concerned with depreciation accounting as it relates to valuation, and the determination of the depreciation of valuation. It would seem to be fair in any given case to give

the method of accounting used by the utility in setting up depreciation allowances its proper effect in determining deductible depreciation or the depreciation of valuation. The foregoing discussion of the several more common methods is given in order that the proper effect of each may be better understood by the valuing engineer.

The Depreciation of Valuation, or Fair Depreciation.

Cost of Decretion.—Finding the cost of decretion is a step in the determination of depreciation, but the result is not necessarily depreciation of valuation. By cost of decretion is meant the loss of service life of an item converted into loss of value. This produces what is sometimes called depreciation and what is depreciation for the particular item. As it may or may not be a quantity to deduct from cost or other base for value when finding value or "fair value", it is not called depreciation here, but cost of decretion, simply to avoid confusion likely to result from the use of the word depreciation in more than one sense.

The first step in estimating the cost of decretion is to make the necessary examination of the property unit by unit. This examination should include an investigation of every element that has any bearing on the future and past service of each item and the character of its maintenance. From the investigation an estimate will be made of the remaining service life of each unit under the method of maintenance being followed; its past service life will be known; from these its total estimated service life, which may or may not agree with assumptions made for past accounting purposes, is found. Examination should show also the efficiency of operation of each unit.

To estimate the fraction of service life consumed requires a knowledge of past service and an estimate of future service life. There are two questions of importance to be discussed in this connection.

1.—Treatment of Inadequacy and Obsolescence.—Experience furnishes a guide in the making of reasonable estimates of service life for those property units devoted to well-known operations, unlikely to be superseded by newly invented units to perform the same service, or to be rendered obsolete or inadequate by the changing requirements of a shifting population, or of commercial methods, or the changing habits and demands of the community. It is very difficult, in some cases, however, to make such estimates for property units which are likely to go out of service for one reason or another before they are worn out.

Nevertheless, it is recognized to be good business policy to make the best estimate possible in the light of such experience as may be available, and to fix a periodic allowance for depreciation reserve accordingly. Indeed, it is prescribed by the Interstate Commerce Commissions and many State Commissions, that the estimated effect

of inadequacy and obsolescence shall be included in the depreciation periodically credited to reserve by common carriers. Many State commissions make the requirement that these causes shall be included, not only in fixing the depreciation credited to reserve, but in fixing the amount of depreciation to be deducted in determining the "fair value" of a property.

Although the matter has not been definitely passed upon by the United States Supreme Court, some Federal and higher State Courts have recognized the propriety of including inadequacy and obsolescence as factors in the determination of depreciation.

The estimation of inadequacy and obsolescence is of necessity somewhat more speculative in character than the estimation of the results of wear, tear, and decay, but there is greater or less speculation in all estimates of future service life of physical property, and it would seem to be inconsistent to consider functional depreciation in estimating depreciation allowances to be charged to operating expense, and then to ignore it in a valuation.

The Illinois Public Service Commission, in the case of City of Springfield *v.* Springfield Gas and Electric Company (March 9th, 1916), in the course of a long discussion and review of the opinions of the Courts on this subject, says:

"Depreciation, accordingly, may be divided into two classes: physical, due to wear and tear; and functional, due to inadequacy and obsolescence which require replacement of equipment before it actually is worn out.

"It is the opinion of this Commission that the weight of authority compels a reasonable deduction from cost new for accrued depreciation —both physical and functional— * * *."

"In view of all facts in this case, the Commission finds that it is but reasonable, proper, and equitable to make deductions from cost new to cover accrued depreciation, both physical and functional, and that it is proper to treat depreciation consistently in all its features. Nothing in the record, and no theoretical consideration presented in this matter, are of sufficient weight to cause the Commission to depart from this reasonable and thoroughly consistent treatment of depreciation, which not only possesses judicial sanction but also is sound in both economic theory and business policy."

The California Railroad Commission, in the case of the Central Pacific Railway Company, decided December 8th, 1915, gives the following definitions which refer to functional depreciation:

"Physical depreciation is caused by the action of the elements and the wear to which a facility is subjected; functional depreciation is caused by the inadequacy or obsolescence of the facility, due to developments which have made it incompetent to perform its function properly or economically.

"Reproduction cost of a railroad less depreciation, in a valuation for general purposes, is the reproduction cost less the diminution in

the value of the physical elements of the property, due to use, age, *obsolescence, inadequacy,* or other causes, plus the increase in value of physical elements due to age or other causes." (Italics ours.)

In the case of the Chesapeake and Potomac Telephone Company of Baltimore City, decided March 8th, 1916, the Maryland Public Service Commission made the following statement:

"Any theory for ascertaining existing depreciation in the plant of a public utility which confines such depreciation solely to the actual, visible, physical, demonstrable deterioration which can be seen by the human eye and measured by the human hand, must of necessity ignore that other species of deterioration which the experience of the past has demonstrated beyond peradventure exists in the property of every telephone company, although it can not always be seen by the human eye or measured by the human hand.

"We refer to that tendency upon the part of all such property to become inadequate or obsolete with the lapse of time. We find it in the case of practically every building which this company owns. Ample, adequate to-day, they may become entirely inadequate to-morrow."

The Colorado Public Utilities Commission, in the case of the Colorado Springs Light, Heat and Power Company, decided March 1st, 1916, made the following statement:

"This Commission is familiar with the decision of the Idaho Supreme Court in Murray *v.* Public Utilities Commission, 150 Pac., 47, and with the theory there announced that there should be no theoretical accrued depreciation deducted from the reproduction cost of the physical property, but the court there states that actual depreciation should be deducted. Just what method an electrical engineer may use in arriving at the actual depreciated value of an electric property without considering theoretical depreciation is somewhat beyond our scope of vision."

A limited search of the practice of public service commissions indicated that inadequacy and obsolescence were included either in ascertaining the amount of depreciation to be included in earnings, or to be deducted from cost to obtain "fair value", by the commissions of the following states: Arizona, California, Colorado, Illinois, Maryland, Massachusetts, New York, Oregon, South Dakota, Wisconsin.

The Board of Public Utility Commissioners of New Jersey, in a decision concerning the rates of the Public Service Gas Company (1 N. J. Bd. P. U. C. 433, December 26th, 1912), distinguished between what is called, in its decision, theoretical depreciation, which must be sufficient to take care of obsolescence and that depreciation which can be determined by observation and inspection, and took the view that, in the future, allowance should be made for depreciation on the theoretical basis, but that as applied to an existing property, where it appears that the depreciation, credited to depreciation reserve in the past,

has not included the effects of inadequacy and obsolescence and thus been collected from the people, nothing should be held against the property for accrued depreciation due to these causes. The decision includes this statement:

"We believe that from this time forth allowance for depreciation should be made where possible, on the theoretical basis, but where depreciation has been charged off, the amount charged off appears to have been not theoretical depreciation, but merely amounts which would measure depreciation ascertained by inspection. We, therefore, conclude that we are on certain ground when the allowance for depreciation which is deducted from the cost to reproduce the property new is the amount representing the wear and tear and aging, and when we do not attempt to estimate the greater amount which would allow for obsolescence and inadequacy."

Although the matter of the inclusion of obsolescence and inadequacy has not been passed upon definitely by the United States Supreme Court, it is possible to infer from the language used in the Knoxville case and in the Minnesota rate case that the Court had in mind as a part of depreciation that due to inadequacy and obsolescence. In the Minnesota rate case the following language is used:

"As the Master said, everything on and above the roadbed depreciates from wear and weather stress. The life of a tie is from 8 to 10 years only. Structures become antiquated, inadequate and more or less dilapidated, * * * cars, locomotives and equipment, as time goes on, are worn out or discarded for newer types."

The use of the terms "become antiquated", "inadequate", "are discarded for newer types" clearly indicates that the master had in view depreciation from obsolescence and inadequacy, and although the Court quotes his statement it makes no unfavorable comment upon these features.

The decision in the Knoxville case contains the following language:

"But the reservoirs, the mains, the service pipes, structures upon real estate, standpipes, pumps, boilers, meters, tools, and appliances of every kind begin to depreciate with more or less rapidity from the moment of their first use."

Since about the only way that reservoirs can depreciate is through obsolescence, their inclusion in the foregoing statement, if not accidental, must have been based on a consideration of obsolescence.

A case in the Federal Court, recognizing the inclusion of functional depreciation in determining the sum to be deducted from reproduction cost, is that of the Des Moines Water Co. *v.* City of Des Moines (192 Fed., 193, 197, Sept. 16, 1911), in which it is stated:

"* * * the question is, What would it cost to-day to reproduce the plant? and from which, to get at the value of the present plant,

there would be deducted the value of depreciation, either by functional or physical depreciation."

In the Kings County Lighting Company case, the New York Public Service Commission for the First District included the effect of inadequacy and obsolescence in the determination of the amount to be deducted from cost for depreciation. When this case was appealed to the Appellate Division of the Supreme Court, that Court decided (People *ex rel.* Kings County Lighting Co. *v.* Willcox, 156 N. Y., App. Div., 603, May 9, 1913) to confirm the action of the Commission, but did not discuss obsolescence or inadequacy.

While most of the opinions of public service commissions and Courts in recent years have favored the inclusion of inadequacy and obsolescence, it is by no means uncommon to find in the opinions of masters and minor Courts, and sometimes of higher Courts, the expression of the view that all depreciation based on the life of a property unit should be excluded, and this would cover the exclusion of inadequacy and obsolescence. The basis in such opinions appears to be largely that, where the service is maintained at 100% efficiency, there is no depreciation. A recent opinion of this kind was that of the Supreme Court of Idaho (Murray *v.* Public Utilities Commission of Idaho, 150 Pac. R., 47, July 1, 1915):

"This Court is of the opinion that the rule of cost of reproduction less depreciation, adopted by the commission, is the correct general rule or principle to be applied in this class of cases. However, we believe that in ascertaining values in this way the worth of a new plant of equal capacity, efficiency, and durability, with proper discount for defects in the old, and the *actual depreciation for use,* should be the measure of value, rather than the cost of exact duplication.

"So far as the question of depreciation is concerned, we think deduction should be made only for *actual, tangible depreciation,* and not for theoretical depreciation, sometimes called 'accrued depreciation'. In other words, if it be demonstrated that the plant is in *good operating condition, and giving as good service as a new plant,* then the question of depreciation may be entirely disregarded." (Italics ours.)

Whitten, Vol. II., page 1150, gives a part of the decision of the Railway and Canal Commission of Great Britain concerning the value of the property of the National Telephone Company and its transfer to the Postmaster-General at the expiration of the Company's license on December 31st, 1911. (16 A. T. & T. Co. Com. L., 491, Jan. 13, 1913.) Whitten makes the following statement:

"Under the purchase agreement between the parties dated August 8, 1905, the purchase price was to be based substantially upon the reproduction cost of the physical property less depreciation.

"In this case the Postmaster-General contended that in computing the life of the plant for the purpose of estimating accrued depreciation,

the life period should be reduced on account of a probable shorter 'effective life' due to inadequacy, obsolescence, or other factors requiring supersession of a particular unit before it has been worn out. The greater stress was laid upon probable supersession through inadequacy due to the probable growth of the telephone business. The court held, however, that to give weight to these considerations involved a confusion of thought and that the possible and even probable growth of the Postmaster-General's telephone business had no bearing upon the present value of the Company's plant. The court, however, attempted to make due allowance for all functional depreciation actually existing in the plant at the time of the valuation."

The trend of opinion at the present time seems to be in favor of including inadequacy and obsolescence as factors in determining depreciation, both as to the sum to be deducted from cost to obtain cost less depreciation and as to the sum to be included in earnings. Until there shall have been authoritative court decisions in this country covering this matter, the estimator should proceed with caution, giving the property the benefit of any doubt, lest by mental speculation he destroy actual, existing value. Where the financial history of the property shows that the charges for depreciation have not included obsolescence and inadequacy, equity seems to require that they should not be included when determining the reproduction cost less depreciation of the property.

Under existing circumstances it is well to segregate the depreciation due to inadequacy and obsolescence from that due to other causes, that the propriety and effect of its inclusion may be weighed by the Court.

2.—Mortality Tables to be Used with Care.—It has been suggested already that remaining service life is determined as the result of examination and investigation; that depreciation is due to several causes, one or more of which may be chiefly responsible for loss of value in any particular case. The purpose of this paragraph is to emphasize this point and to suggest that ordinary mortality tables for physical properties are not to be carelessly used in this connection. The determination must be based on an investigation which includes not only age but character of past, present, and probable future service, character of past, present, and probable future maintenance, possible apparent or probable approaching inadequacy or obsolescence; and often the accounts must be examined to find the amount of money spent in repairs and general upkeep, and the regularity of the expenditure. Not only these but experience with other like properties under similar conditions should be considered, and sometimes properly prepared mortality tables giving estimated service life may be useful.

As has been suggested, many properties under proper maintenance will last forever, so far as human intellect can comprehend the meaning of that term.

That this is practically true, present existence of the following named structures seems to prove:

Main stone sewer, Rome	Built	600 B. C.
Arch bridge, San Angelo, Rome	"	200 A. D.
Several stone bridges at Nuremburg	"	1486-1728
Wooden bridge (Kapollsbrücke)	"	1333
Iron suspension bridge, Nuremburg	"	1824
Westminster Abbey, London, old portion stone—Built		13th Century
Hatfield House, England (Marquis of Salisbury)	"	1611—Stone.
Saint Lawrence Church (Nuremburg)	"	1477— "
South Church, Boston	"	1750—Brick.
Church, Alexandria, Va	"	1784— "
Howland House, Plymouth, Mass	"	1660—Frame.
Windmill, Nantucket, wood and shingles	"	1746— "
Washington's home, Mt. Vernon, Va	"	1784— "
Hobson's House, Montgomery, Ala	"	1815— "

On the other hand, apparently permanent structures have but a short life; what were new high buildings only a few years ago are already destroyed to make way for yet higher buildings. This means inadequacy and obsolescence. It shows that mortality tables based on the older structures are not applicable to the modern structures, and, based on averages, they would be likely to be far from the truth in any particular case.

At a recent meeting of railroad equipment officers from all sections of the United States it was determined from railroad records that:

"Obsolescence and inadequacy have been the cause of retiring practically all equipment in this country, and wear and age have not, except in very few cases, been the cause of retirement. There is, therefore, little data on which to base life tables, and as wear and age have not been the cause of retirement, experience is lacking which would permit the preparation of such life tables."

Although data may be lacking on which to base estimates of future loss of service life due to age and use, it would seem that, with all the data available with respect to inadequacy and obsolescence, there might be sufficient to estimate in a general way what the service life of railroad equipment is under varying conditions of service. Indeed, some such estimate must be made for any given road, in order to fix reasonable depreciation allowances for equipment as required by the regulating body. An estimate prepared with care for any given unit from a large number of data covering the service life of similar units would certainly be more trustworthy than one based solely on an examination of the particular unit. Tables showing the lives of certain units will be found in Appendix II.

Depreciation of Overhead General Expense.—Shall depreciation be estimated on overhead and general expense? The Committee believes

that the answer depends on the character of those expenses, and divides them into two classes: (1) Those which, like promotion expense, preliminary and location surveys, legal and other expenses, connected with the creation of the property, cannot be segregated into parts assignable to particular units, but apply to the property as a whole; (2) Those which would be incurred were the units to be reproduced, not as a part of a general reproduction, but for the maintenance of the property.

Expenses of the second class may be included sometimes in unit prices, but are not always entirely separable. Expenses of the first class cannot be thus included. The test as to whether or not a particular item of overhead shall be depreciated is: Will it be incurred in replacing units that are retired. If so, it is part of the cost of those items the service life of which is being consumed, and hence will be a part of the sum to be depreciated with the loss of service life of those items. If not, it is a part of the ever continuing property, and can suffer no depreciation except as the exchange value of the whole property may be lowered, in which case everything that went to make it loses value.

The Committee is of the opinion that all overhead charges which may be included in the unit prices of particular units, or which are specific charges against particular structures, should be depreciated, if the particular units or structures are to be depreciated, and as a part of the cost of those units or structures; that all overhead charges which would be incurred in replacement of units but which may not be easily separable in a reproduction cost estimate, should be depreciated by the average depreciation assigned to the units to which they belong; that overhead charges appertaining to the property as a whole and not to be reincurred in the replacement of units should not be depreciated at all, if the property is a continuing one.

Depreciation of Valuation Affected by Accounting Methods.—Decretion or loss in service life being always present in any going property, the cost of decretion is always present. Decretion is a fact, and is independent of methods of accounting, or character of regulation, but is much dependent on character of maintenance. In determining it, one is not helped in any way by reference to the accounting methods. He must examine the property units, estimate their remaining service lives, know their elapsed service lives, the character of their past service and maintenance, and the prophecy as to future service and maintenance. To estimate the cost of decretion one must know, also, the costs or estimated reproduction costs of the property units, but the amount of the estimate and whether or not it shall be treated as depreciation of valuation may be, and very probably will be, dependent, at least in part, on methods of accounting for depreciation

that have prevailed. Whether or not it will thus depend may also depend again on the character of public regulation.

(*a*)—Replacement Method.—If by order or sanction of a regulating body, or by long continued proper custom under no regulation, a property has been maintained in normal working condition, necessarily less than new in some or all of its parts, by the replacement method, and at any given date is being valued for any public purpose and at that date shows normal condition, all its several parts being in as good condition as could be expected, the accounts showing that always those amounts have been expended in renewals that were necessary to keep the property in normal working condition, and the fact appearing that no expenditure reasonably to be expected could put the property in better than the normal condition in which it is found, and that no unusually large expenditure is presently to be necessary for this purpose, then, in spite of the fact that there is an existing decretion in its several parts, there should be found no depreciation of valuation. Under the method of accounting, the public has not paid, and could not pay, for the accrued depreciation, and under this condition its accrued obligation to pay should be considered an asset of the company owner.

If parts of the property are maintained under the replacement method and part by some proper allowance method, then, except as noted below, depreciation of valuation should be found with respect to those parts maintained under the allowance method, but this depreciation of specific physical units will be made good in whole or in part by existing funds or property purchased with allowances, either or both of which will be included in the valuation as they are found.

Railroads are properties of the class just considered. Their owners have long maintained the greater part of their perishable properties by the replacement method, and, except for a comparatively recent requirement of allowances or reserves for equipment, this method has been sanctioned by the Interstate Commerce Commission. Therefore, for such properties as are found in normal condition, as suggested above, there will be no depreciation of valuation for those items maintained by the replacement method. If these items are found below normal working condition, there will be depreciation of valuation, sometimes called deferred maintenance, and it will be measured by the sum necessary to bring the items to normal working condition—not new, the initial decretion necessarily present in such items in normal working condition being ignored, but upon restoration to normal working condition, the integrity of the investment should be considered to be renewed.

If in the judgment of the valuing engineer, the replacement method may not be used with propriety for a given property, either because not in accordance with law, or because the method is not adapted to the property, then whether or not the property has been maintained in the

past under this method the valuing engineer should estimate depreciation in the amount of the cost of the decretion he finds. There can be no certainty that the property will be properly maintained in the future.

When a comparatively new property, other than a railroad, is to be valued, and it has not been under any regulation that has affected its accounting methods, the law as laid down in the Knoxville decision would seem to make it necessary to find depreciation of valuation in amount equal to the cost of decretion found for all items, whether or not maintained by the replacement method. The Committee, however, believes that this may work hardship and injustice in some instances, and suggests that in such cases the facts be reported with such recommendations as to equity, as may seem fair to the engineer.

The public utility is entitled to see that from earnings the value of the property invested "is kept unimpaired" and that "before coming to the question of profit at all, the company is entitled to earn a sufficient sum annually to provide, not only for current repairs, but for making good the depreciation and replacing the parts of the property when they come to the end of their life". This is a reasonable right, the existence of which permits of renewals of parts of a property, that can be called current repairs, under the replacement method.

(*b*)—Allowance Methods.—If either the straight-line or sinking-fund theory has been used in computing depreciation and the method of accounting for it has been prescribed by a regulating body or voluntarily followed by a company owner from the beginning, the same theory, so far as it applies to the property in question, should be used for estimating the cost of decretion, and the entire cost so found lessened by any accumulated depreciation funds will appear as depreciation of valuation, unless the sinking-fund method of accounting has been used. In the latter case, if the valuation has to do with the reasonableness of the return and the accounting is to go on as before, apparently-existing depreciation would not be *depreciation of valuation,* and therefore would not be deductable; but if the valuation has to do with condemnation or purchase, then, as in other cases, the apparently existing depreciation is depreciation of valuation, and the owner should receive the depreciated value of the physical property and the existing fund.

As has been said, the unit-cost method of estimating depreciation can hardly be used with convenience as an accounting method. When there has been no regular depreciation accounting method followed, the unit cost method for estimating depreciation at any given time may be used with fairness.

The Committee has advised that the proper interest rate to be used in computing depreciation annuities is the same as the fair rate of return. This advice pertains to present and future accounting.

If, however, an old property, that has been properly maintained under the sinking-fund theory, is to be valued, that rate of interest should be used in computing the growth of theoretical depreciation that has actually been realized and has produced the existing depreciation fund or reserve. If there has been no depreciation accounting and it is desired to estimate the accrued depreciation under the sinking-fund theory, the engineer may choose the interest rate, but, inasmuch as new accounting to be installed after the valuation or rates to be based on the valuation should include, in accordance with the Committee's advice, an interest rate for depreciation equivalent to the fair return rate on capital, it is suggested that this same rate be used in computing the accrued depreciation.

Effect of Investing Allowances in the Property.—Under the straight-line or compound-interest method of accounting it is contemplated that, so far as possible, the depreciation allowances shall be invested in additions or betterments to the property. If such contributions of the public toward depreciation of existing property are invested in betterments and additions, then there must be depreciation of valuation to whatever extent cost of decretion is found, because in the inventory the betterments and additions made with the depreciation allowances will be listed as part of the property to be valued just as are the original items, the depreciation allowances for which were used to provide the betterments or additions. There will be no offsetting funds, but there will be property contributed by the public which cannot be capitalized and for which credit can be given to the public only by deducting an equivalent depreciation.

If, at the time of any valuation, not all the contributions of the public toward depreciation have been re-invested in the property, depreciation of valuation should still be found in the amount of the estimated cost of decretion of the physical property, but this will be offset to such extent as other assets, such as cash or outside securities, are in hand representing the portion of allowances, not re-invested in the property, but held for this purpose, and which, therefore, should be inventoried and included in the valuation.

Effect of Regulation on Depreciation of Valuation.—Regulation which determines the method of accounting will in part determine the amount of depreciation of valuation when finding "fair value", because it determines the method by which the public shall pay for the loss of service life.

It has been said by the Courts that it is not possible to go into the unregulated past to find losses or excessive profits, but if there has been regulation determining the method of accounting for depreciation from the beginning, or for a long time, and this regulation has fixed the sums to be included in the earnings for depreciation, it should be permissible to inquire into the adequacy of these sums. It seems

clear that in this respect it is proper to go back into the past and inquire to what extent the public has compensated for the loss of value due to loss of service life. If the regulating body has prescribed the replacement method for the whole period that units have been in existence, then, although depreciation may exist, it is not depreciation of valuation, because, under the method of accounting, the public has not paid, and could not pay, for the accruing depreciation, and is still under obligation to pay for it.

Methods of accounting in force at the present time which make proper provision for the accruing depreciation should not have full weight, if, in previous years during the life of the property units, other methods were in use which did not make proper provision for such depreciation. The amount of depreciation of valuation in such cases should be equivalent to the accumulated contributions of the public for depreciation allowances under the various methods of accounting which have affected the property unit from time to time. The public is still under obligation to make good that part of the loss of service life not yet paid for, and this obligation should be considered to be as much the property of the company usable to offset accrued depreciation as renewal funds or property actually in existence.

Whether or not this reasoning will stand in any case is for the Court to determine. Valuing engineers and accountants should report what they find as to the actual cost of decretion, and the sums which have been received to offset such cost, under the methods prescribed by the regulating body from time to time. They may give their opinions as to the amount of depreciation that should be deducted from cost to find cost less depreciation as an element in the quantity known as "fair value".

If regulation has not fixed accounting methods, but has limited the earnings, it should be permissible to inquire whether the limited earnings have been sufficient to pay operating expense, depreciation, and fair return. If so, depreciation found should be considered depreciation of valuation to the extent warranted by the accounting methods lawfully or properly followed; if not, a question arises. It is remembered that the duty of the company owner is first to maintain the property "before coming to the question of profit at all", and that it is the duty of the regulating body to see that rates are such as to permit the company owner to earn operating expense, depreciation, and fair return. If the regulating body has made sufficient earnings impossible, is it still the duty of the company owner to maintain the property before paying fair return to its security holders? If it is, depreciation of valuation should be found in the amount of the total cost of decretion or so far as warranted by the accounting methods followed. If not, depreciation found should not be considered depreciation of valuation except to the extent covered by earnings after

deducting operating expense and fair return. This is a matter of equity to be determined by a Court, and the Committee will not venture an opinion. If under regulation the property is a losing venture, it is not included in the class of properties now being considered. The depreciation existing will be found and reported, together with all pertinent facts, and the Court will determine the equities of the case.

Concluding Suggestions.—The valuing engineer should bear in mind that when a company owner has invested a reasonable sum in a property for public service, it is entitled to, but not guaranteed, a fair return on its investment, so long as the money remains in the property, either as property, or funds, or accrued public obligation to pay. Therefore, so long as the company owner keeps a sum equivalent to the total investment at work for the public, either as property serving the public, or funds held in reserve for such property, no policy should be followed in estimating depreciation that will reduce the property to a value less than the investment, or, when using cost of reproduction less depreciation as a basis of "fair value", to a value less than the cost of reproduction of that part of the property estimated to have been created with company funds, acquired by gift or in any way not the result of public contributions to cover depreciation.

Valuing agencies should also bear in mind the fact that for 30 years, until 1909, the Supreme Court of the United States not only recognized the replacement method as a proper method for the maintenance of public utility property, but disapproved the practice of charging the public with depreciation allowances in advance of the necessity for their use, holding that only money actually expended could be charged in operating expense and so collected from the public. In 1909 the Court apparently reversed itself in the Knoxville case. For the purpose of the present discussion, there is no need to speculate as to the strict applicability of this decision to all kinds of properties or property units, but it is thought to be proper to advise that valuing agencies should proceed in such a manner with respect to depreciation of properties that grew up under the law as it was before 1909, so that such properties shall not be unfairly reduced in value by strict application of a current interpretation of the Knoxville decision to all their parts. If the law changes, equity would seem to demand that the change shall not be retroactive; and it may be said fairly, we think, that though law must in general govern in all matters on which it speaks, yet, with respect to such matters as the details of valuation of public utilities, of which depreciation is one of importance, in the determination of which there never can be more than approximate results, and concerning which the minds of men have been and are changing, equity should not be ignored.

Appreciation.

Definition and Application.—Appreciation, as used in connection with the valuation of public service properties, according to the definition given in the Glossary "is generally restricted to physical items, and measures their gain in value due to age, use, and properly directed labor. Its principal application in connection with railroads is to roadbed, which, for instance, is increased in value by solidification and grassing of slopes."

Appreciation, therefore, represents the improvement in quality and usefulness of certain parts of the physical properties of a railroad or other public utility property, and it results from the lapse of time from work not specifically charged to capital account, from maintenance, from use, etc., and covers items not represented either by the quantities, or unit prices, that are determined in connection with a valuation.

Upon a railroad, appreciation is found in connection with roadbed, both in cuts and fills and in the modification of embankments; the general improvement of the slopes through various forms of protection of both embankments and cuts; and improvement of drainage facilities along the roadbed, and right of way in general.

In the case of irrigation works, appreciation may be realized upon dams, canals, and embankments, particularly when not originally built with great care.

Its determination is difficult, and care must be exercised lest any of the allowances be duplicated in development expense.

General Principles.—In the case of the San Joaquin, etc., Canal Co. *vs.* Stanislaus County, *et al.*, 191 Fed., 875, in which was involved the value of certain canals, the Company claimed that the earthwork had appreciated in value, the claim being that, by the lapse of time and by reason of the packing of the banks and the silting of the canals, an additional value had come, in the way of avoiding breaks and in preventing the loss of water by seepage. Judge Morrow, in reviewing the findings of the master, quoted from various Supreme Court decisions to the effect that a public utility property is entitled to earn a fair return upon the reasonable value of its property, at the time it is being used for the public, and added:

"We then have this rule for ascertaining the value of a plant of the character of that owned by the complainant in this case: Find the cost of reproduction as of the date of the use in question and then from this cost deduct the depreciation that has occurred from age and usage."

The master to whom the case was referred having determined that depreciation should be deducted, said:

"Likewise it would seem to me that if by reason of a lessened amount of depreciation, the canals of complainant in their present

condition are capable of delivering more water, thereby producing greater revenues to the Company, it would be of more value than a canal estimated on the basis of the reconstruction figures aforesaid."

The master, however, was not satisfied with the evidence submitted as to the amount of depreciation or appreciation, and held:

"After a careful examination of all the testimony on this question I find that I am unable to make either a calculation as to appreciation or depreciation of the earthworks of the canal and shall assume that the one offsets the other."

Circuit Judge Morrow, when he reviewed the case (page 855), stated:

"After carefully reading the testimony on this subject I have reached the same conclusion the master did with respect to this claim."

In this case, therefore, appreciation was acknowledged; but the one was considered to offset the other.

In the Minnesota Rate case, where the value of railroad property was sought to be determined through the cost-of-reproduction method, and where the Court below found that appreciation and depreciation existed in practically equal amounts, so that the one offset the other, Justice Hughes stated, in settlement of the matter:

"If there are items entering into the estimate of cost which should be credited with appreciation this also should appear, so that instead of a broad comparison there should be specific findings showing the items which enter into the account of physical valuation on both sides."

It appears from this decision that there can be no general offsetting of appreciation against depreciation, but that appreciation must be determined separately from depreciation.

Determination and Estimation of Appreciation.—It is now conceded that all items that are measurable by units, such as rip-rap, protection of slopes by sodding or other forms of vegetation, in whatever manner it has been secured, should be measured and unit prices attached, and the value of rip-rap, sodding, and other forms of slope protection should be determined in that manner; and that this method should be followed in valuing common carrier properties, and other public service properties, such as dams, protected embankments of reservoirs, etc.

There are two forms of appreciation that are particularly applicable to railroad properties; one is described by the terms "Solidification and Seasoning", and the other by the term "Adaptation".

Solidification and seasoning are found in railroad, canal, and other embankments, and along the slopes of railroad cuts. They are produced by compacting under the action of the elements; through use, such as the running of trains of a common carrier; through the skillful

direction of labor, of a common carrier or other public service corporation, in checking wastes on the slopes of embankments, and encouraging the growth of vegetation:—the final result being a solidified condition of the entire roadbed or other form of construction with greater compactness in the case of banks, and the drying out of cuts and the solidification of the slopes of cuts. The results of solidification and seasoning are a reduction in the cost of maintenance of the property of any common carrier, or the property of any public service corporation that receives any form of protection due to age and the action of the elements, combined with properly directed maintenance labor. Rip-rap becomes more settled, compact and secure; trees and shrubs, planted for ornamentation or snow protection, increase in usefulness and value with age and care. In railroad property a higher value is evidenced by the fact that trains can be run at greater speed and with greater safety; and that the expense of operation is reduced, and fewer accidents occur, after many years have passed since the property was a completed new one.

It is generally conceded that appreciation in the form of solidification and seasoning continues over a long period of years. For example, a railroad 40 years old is much better than one 5 years old. In measuring all forms of appreciation which may occur, however, it is necessary to have a full knowledge of what methods have been used by common carriers in process of construction and in the period of physical development following the construction period. A railroad property is not a finished product when the construction forces have put its parts together and turned it over to the operating department. The subsequent work done upon it, and the quantities subsequently added to it to produce the finished product, should be covered by the cost of reproduction rather than under appreciation.

Take, for example, the Virginian Railway, 444 miles long; tracklaying was started in 1905; train service was begun on some parts of the line in 1907; operation was begun under difficulties in 1910; ballasting, etc., was completed in 1915. In the 8 years (1907 to 1914, inclusive), during which physical development period the first stage of solidification was in progress, the excess cost of ordinary labor per mile of track totaled $1 630.

Again, on the Big Sandy low-grade line of the Norfolk and Western Railway, 59 miles long, tracklaying was started in November, 1903, and operation was begun under difficulties in 1904, with some construction work in progress until 1907. During the 3 years of the physical development period, while some construction work was still under way and the first stage of solidification was in progress, the excess cost per mile of ordinary section labor was $3 090 more than that during the 3 years immediately following.

The items of cost in general maintenance labor, such as these incurred during the period between the beginning of the operation of any portion of a railroad and the final completion thereof by full ballasting, etc., which generally covers a period of several years, is an item of cost to the property, whether it is charged to construction accounts or not; and it most frequently has not been so charged in the past. Hence, in determining the cost of reproduction new of a property, this item of cost, which will vary in different sections of the country, as shown by the two examples, the extreme case given being in a difficult country, so far as control of slopes is concerned, must be added to the cost of reproduction as determined by measurable units. Care should be taken in analyzing these costs that they may not be duplicated in reproduction cost or appreciation and in development expense.

In addition to this, however, there is an increased value that comes to such a property by reason of solidification and seasoning, due primarily to time and use, that continues for many years after the entire completion of a railroad.

Adaptation.—Adaptation is the adjustment of the physical property to its environment and purposes, and is brought about in the course of time by the action of the elements combined with labor. For example, in the case of a stream, where by successive floods the stream may deepen and scour out its channel and make more safe adjacent embankments. After a railroad is in operation, many changes are made in roads and drainage, as between the railroad and adjoining lands, all traces of which are lost in a few years thereafter; likewise, culverts originally built are found to be too small and are necessarily enlarged, or the approaches and outlets are changed. Briefly, therefore, adaptation is exemplified in improvement in drainage, both on the right of way and outside the limits of the right of way, which has been made from time to time by the forces of the railroad company or other public service corporations, the value of which cannot be determined by measuring items of quantity at the date of valuation. It is an item of charge to the cost of reproducing a roadbed or dams, canals, and embankments of the class mentioned, of the drainage area in connection with the dam of a water company. This class of appreciation should be taken into consideration along with solidification and seasoning.

The best method of estimating appreciation due to solidification, seasoning, and adaptation appears to be by comparing the cost of maintaining a new road, directly after it has been entirely completed, with the cost of maintaining an old road in the same district under similar conditions of traffic.

In one case of a western railroad the cost of maintenance of a new line, 175 miles long, was compared with the cost of maintenance of an

old line of the same length. In 2½ years the excess cost spent on the new road, to make it comparable with the old one, aggregated $1 400 per mile.

Another way is to assign a direct percentage to the grading quantities.

In California allowance was made of 2% a year for 5 years on all earth and loose rock quantities, and 1% per year on all solid rock quantities, for solidification and seasoning. This might be used as a minimum, to which should be added $600 per mile for all roads more than 5 years old.

Care must be taken in estimating appreciation that items of labor and expense, included therein, may not be duplicated in development expense.

CHAPTER VII.

DEVELOPMENT EXPENSE.

The Nature of Development Expense.

The Committee recognizes two principal periods in the creation of a going concern operating a physical property for the production of a commercial commodity, be that commodity water, gas, electricity, sugar, textiles, or any other thing, or the transportation of persons, property, or thought. These two periods are:

1.—The construction period, which covers the interval from the inception of the enterprise to the beginning of regular operation.

2.—The development period, which covers the interval from the beginning of regular operation to the time when efficiency and profitableness of operation shall have been achieved.

Corresponding with the two periods recognized, the Committee divides costs into two classes—construction costs and development expense. The estimated cost of construction includes all costs incurred during the construction period, and, as will appear, may include certain costs incurred during the development period.

Development expense is of two kinds: (*a*) the cost of tuning up the physical property and bringing it to present operating efficiency; and (*b*) the cost of acquiring the business.

The cost of tuning up the property and bringing it to its present state of operating efficiency stops at the end of what may be called the physical development period, which is usually included within the development period as previously defined, and the cost may be included in the cost of construction.

The cost of acquiring the business usually extends throughout the development period, though there are cases in which a profitable business develops at the very beginning.

The physical development cost, which should be included as a part of the construction cost, comprises the items of work required to bring the property to an efficient operating condition, such as: the extra work required on new embankments to maintain tracks in proper operating condition; the cost of testing and sometimes of re-adjusting and remodeling all sorts of machines to make them operate properly, or to increase their efficiency; the making good of defects which have become visible after a property becomes operative; and a great variety of other expenses incidental to the first operation of a property, which are not incurred in subsequent years after that same property has reached full operating efficiency.

The Committee also recognizes going value, as defined by it, as quite independent of development expense, and discusses it in Chapter VIII on INTANGIBLE VALUE.

DEVELOPMENT EXPENSE AS A PART OF ORIGINAL COST.

On the basis of original cost, the development expense in any one of the early years when the earnings are deficient will be obtained by deducting the actual receipts from the gross amount that the owner of the property is entitled to earn in that year. Such gross amount is the sum of these items:

(*a*) The expenditures for operating expenses and taxes;
(*b*) A sum to offset the waste of capital through depreciation;
(*c*) A sum representing a fair return on the investment, including money, and the money value of services and of other considerations.

The difference between the actual and the fair gross receipts for the first year is the development cost for the year, which should be added to the investment at the beginning of the second year in order to obtain a basis for estimating fair return for that year. By following the same course for successive years, adding the development cost for each year until there is no further deficiency of earnings, a sum will be reached which will represent the amount of the investment at the end of the development period, and the difference between the investment at the beginning of operation and at the end of the development period will represent the development expense.

It should be noted that the development cost obtained in the manner recommended includes also the physical development costs of those years when the earnings were deficient. So far as possible, these should be separated from the total sum found, and included as a part of the construction cost of the physical property.

After the property originally built has attained adequate and steadily maintained earning capacity, the ordinary additions of subsequent years may involve little or no additional development expense, except for the cost of testing and tuning up plant units, because there will be no deficiency of earnings caused by such additions, but, from time to time, additions may be made of such magnitude that the earnings will again become temporarily deficient.

Losing Venture.—Fair dealing demands that the development expense actually incurred in tuning up and developing the business of a normal property be properly recognized, and the Committee believes there is no more equitable way of doing this under normal conditions than by including it as a part of the capital account. It recognizes, however, that there may be abnormal conditions under which this method should not be followed, as, for instance, where the net

receipts are unnecessarily low on account of bad management, or where bad judgment has been used in the location or design of the plant, so that the property is bound to be a "losing venture".

It does not seem desirable that the general rule should be modified because of these exceptions. In the application of the rule to specific cases, it will not be especially difficult to distinguish between profitable properties and distinctly losing ventures, but it may be difficult to distinguish legitimate development expense from the cost of bad management, in connection with those properties that are neither very profitable nor distinctly losing ventures.

Amortization v. Capital Charge.—It is sometimes urged that, although development expense is a real expense incurred in developing a property and its business, that part which is not a construction cost should be separated from other costs and repaid by means of earnings in excess of a fair return on the "fair value" of the property as soon as practicable after the business has increased to such an extent that excess earnings may be obtained by the maintenance of the previously existing rates. This requires the maintenance of the high initial rates for a longer period than would be necessary if the development expense is included in the return base.

When it is thus included, the deficiencies in the early years are made good by the greater earnings throughout all subsequent years, resulting from the larger return base on which earnings are predicated. The inclusion of development expense in the return base appears to have advantages in that it furnishes a more definite and simple method of accounting, and permits the adoption of normal rates at an earlier period than under the other plan. The cost of developing the business is just as much a part of the cost of producing the property, as is the cost of the physical items. Moreover, the Committee sees no reason, in principle or in equity, why this element of cost should be amortized, any more than the cost of the physical property.

Difficulties.—In the determination of development expense there may be difficulty in determining what is a fair return, not only because of possible differences of opinion as to what is a proper percentage to adopt as a return on investment, but because of the difficulty in determining a proper sum for the depreciation allowance to be used in computing the gross fair earnings. This difficulty is minimized if the attempt is made to determine these sums only for the first few years of the life of a property, when there is a deficiency of earnings, and is greatly increased if the attempt is made to determine the fair return and the proper depreciation allowances for a series of years, while the deficiencies are being offset. Practical attempts to determine the net excess or deficiency of earnings in a long series of years, in the case of properties of considerable age, have shown that a slight difference in the percentage assumed for the fair return will cause such very

great variation in the estimated development expense as to show that the method will not give trustworthy results when applied to most existing properties.

The Committee recognizes that public regulating bodies having control of both the accounting and the rate-making of the utility from the beginning of its operation may in the future adopt other methods which will offset such losses or amortize them, but these methods should not be applied to existing properties.

The Courts have ruled that the effects of regulation cannot be made retroactive, and that one cannot go into the "unregulated past".

Ames *v.* Union Pacific, 64 Fed., 165, 176 (1894). Mr. Justice Brewer. "The same rule controls if a railroad property is sought to be appropriated. No inquiry is open as to whether the owner has received gifts from States or individuals, or whether he has as owner managed the property well or ill, or so as to acquire a large fortune therefrom. It is enough that he owns the property, has the legal title; and so owning, he must be paid the actual value of that property. If he has done any wrong in acquiring or using the property, that wrong must be redressed in a direct action therefor, and cannot be made a factor in condemnation proceedings. These propositions in respect to condemnation proceedings are so well settled that no one ever questions them. The same general ideas must enter into and control legislation of the kind before us [rate regulation]."

Public Service Gas Co. *v.* Board of Public Utility Commissioners. N. J., 87 Atl., 651 (1913), which was sustained by the Supreme Court of New Jersey, 87 Atl., 657 (1913).

"But as we have indicated above, the business thus acquired must be regarded as a legitimate part of the property of the company. We cannot equitably project back into the unregulated past a nome of prices that might to-day be regarded as fair and adequate, and assume that actual rates exacted in the past, in so far as they exceed what are now deemed fair, have not lawfully become the property of the company. If these high rates in the past have been employed by the company to acquire an intangible property in the shape of extensive patronage, that expectation of patronage is theirs and on its fair value the company is entitled to a return. It may or may not be a subject of regret that regulation was so long deferred; but deferred regulation is no excuse for refusing at present to allow a fair return upon what is the lawful property of the company."

Therefore the Committee holds that development expense cannot fairly be assumed to have been amortized in the past in old properties by excessive earnings, and that it should be included in present valuation, whatever changes in policy as to future costs or increments in value may be made by the regulating body hereafter. Moreover, the Committee thinks that in future regulation it is sounder public policy not to amortize development expense.

The determination of the proper rate of return is a matter to be decided by the Courts or regulating bodies, and an engineer, in making an estimate of development expense in the manner recommended, should state the rate of return on which it is based, so that the Court or commission will have the means of modifying the estimate.

Development Expense as a Part of Reproduction Cost.

The foregoing discussion relates to development expense as a part of the determination of the original cost of a normal property, and, as it is a real cost in connection with the original property, it is also a proper element to include in reproduction cost, but the difficulties of determining the development expense of reproduction cost are greater than in the case of original cost, because of the assumptions which it is necessary to make.

Shall it be assumed that the development expense will be based on the conditions which existed when the various parts of the property were created, or on the expense that would be involved if the property were to be re-created under the conditions existing at the time of valuation? Shall it be assumed that the existing business is ready and waiting, or that it has to be re-acquired at the time of valuation?

If the property under consideration has grown by additions made from time to time, so that it is now several times as large as when originally constructed, there would be, on the basis of the conditions which existed when the property was created, a large percentage for development expense in connection with the original property, because there had to be an actual acquisition of business at that time; but, on the other hand, there would be a very small percentage of development expense attributable to the additions.

If based on reproduction under existing conditions, the development expense would apply to the entire property reproduced.

The Committee believes, as already indicated, that development expense is a real cost in the production of a going property, and that it should not be ignored in the reproduction cost. It is somewhat difficult of determination in any case, by reason of the necessity of distinguishing between proper deficits and deficits due to management, in ordinary properties which are neither very profitable nor losing ventures, but no such difficulty of estimation should prevent its inclusion, when present, and when its amount is estimated as well as possible. It is especially a feature of valuation which should depend on reasonable assumptions based on known occurrences in connection with the property under consideration or similar properties in other cases.

Attitude of Courts and Commissions.

Development expense has been generally recognized in recent years by both Courts and commissions, usually under the title of "going

value". Sometimes these bodies have used the terms "going value" and "going concern value" to represent the intangible values of a going concern as compared with a dead one, but many times the terms are used with a meaning corresponding to the definition of development expense given by the Committee.

Going Value (Development Expense) Discussed.—A very complete discussion of going value is given by Judge Miller of the New York Court of Appeals in the Kings County Lighting case (People *ex rel.* Kings County Lighting Co. *v.* Willcox, 210 N. Y., 479, March 24, 1914). He gives (at page 492) the following definition of going value, which agrees closely with the definition of development expense adopted by the Committee, and the decision, although using the term "going value", may be considered as relating to development expense.

"I define 'going value' for rate purposes as involved in this case to be the amount equal to the deficiency of net earnings below a fair return on the actual investment due solely to the time and expenditures reasonably necessary and proper to the development of the business and property to its present stage, and not comprised in the valuation of the physical property."

This case was one in which an appeal was made from the Court below, and four questions were asked. The one relating to development expense was as follows (at page 481):

"(1) Was the relator entitled, upon the facts shown in the record, to have the commission make an allowance for going value in determining the value of the relator's property used in the public service?"

The opinion is too long to reproduce in full in this place, but the more pertinent parts are as follows:

"It is now generally recognized that 'going value', as distinct from 'good will', is to be considered in valuing the property of a public service corporation either for the purpose of condemnation or rate making, but there is a wide divergence of view as to how it is to be considered."

* * * * * * *

"The difficulty of determining 'going value' will not justify the disregarding of it. Rate making is difficult. But that will not justify confiscation. The difficulty, however, will lessen, as it does in most cases, when we cease to think about the subject vaguely. What then, is 'going value', and how is it to be appraised?

"It takes time to put a new enterprise of any magnitude on its feet, after the construction work has been finished. Mistakes of construction have to be corrected. Substitutions have to be made. Economies have to be studied. Experiments have to be made, which sometimes turn out to be useless. An organization has to be perfected. Business has to be solicited and advertised for. In the case of a gas company, gratuitous work has to be done, such as selling appliances at less than a fair profit and demonstrating new devices to induce

consumption of gas and to educate the public up to the maximum point of consumption."

* * * * * * *

"To view the matter in another aspect, take the case of a public service corporation with a plant constructed just ready to serve the public. It is going to take time and cost money to develop the highest efficiency of the plant and to establish the business. Three courses seem to be open with respect to rate making, viz.: 1, to charge rates from the start sufficient to make a fair return to the investor and to pay the development expenses from earnings, a course likely to result in prohibitive rates except under rare and favorable circumstances; 2, to treat the development expenses as a loss to be recouped out of earnings, but to be spread over a number of years, in other words, as a debt to be amortized, that involves complications, but would seem to be fairer to the public and certainly more practical than the first; 3, to treat the development expenses, whether paid from earnings or not, as a part of the capital account for the purpose of fixing the charge to the public. The last course would seem to be fairest to both the public and the company, as well as the most practical.

"It may be, as is urged, that a well-conducted enterprise will charge the cost of developing the business to operating expenses, and that it would open the door to an overissue of securities to permit the capitalization of early losses. In answer, it is sufficient to say that we are dealing, not with proper methods of bookkeeping, not with the proper capitalization upon which to issue securities, but solely with the fair return which the company is entitled to receive from the public. Treating a reasonably necessary and proper outlay in building up a business as an investment for the purpose of determining the fair rate of return to be charged is far from holding that it should be treated as capital against which securities might be issued.

"We do not say as matter of law that the third course above outlined should be adopted as an original proposition. That may present a question of economics, depending on the particular conditions involved. The commission in this case had to determine the rate to be charged, not by a new company with no business, but by an old company with an established business. The first question, therefore, to determine on this branch of the case was whether the company had already received a fair return on its investment. If it had received such return from the start, or if in later years it had received more than a fair return, the public would already have borne the expense of establishing the business in whole or in part, and to that extent the question of 'going value' for the purpose of fixing a present rate would be eliminated; for it must constantly be kept in mind in dealing with this problem that the company is entitled to a fair return and no more. If it has already had it, that is the end of the matter. If it did not receive a fair return in the early years owing to the establishment of the business, a subsequent rate must allow for that loss or it will be confiscatory."

The conflict in view, expressed in the concluding sentences quoted above, and the earlier citation from the U. S. Supreme Court, in Ames *v.* Union Pacific, 64 Fed. 165, 176, (1894), is to be noted.

Nevertheless, the Federal Court decision has been reaffirmed in a long line of later decisions, and would seem still to hold, as the position of the highest Court of the land.

Conclusions.

The Committee recognizes development expense as a possible, usual, and, in most undertakings, an unavoidable real cost in the production of a normal going property, and as quite distinct from going value as defined by it, and discussed in Chapter VIII upon Intangible Values. It is measured in nearly all cases by the difference between the amount which the company is entitled to earn in the early years and the amount which it actually does earn.

On the basis of original cost, if the accounts are available, the development expense can be determined from the accounts, taken in connection with estimates of the proper sums to be allowed for depreciation and for fair return on the investment. If the accounts are not available, the development expense must be estimated on reasonable assumptions based on known occurrences in connection with similar properties. It is commonly the case that the general conditions existing when the original property was created can be ascertained with sufficient accuracy for determining whether or not there were early deficits and approximately their amount.

For ordinary additions to the original property, in the case of a prosperous company, there would be no deficiency of earnings, but, from time to time, large additions might cause a temporary deficit which should be taken into account.

As development expense is a real cost of producing the original property, it should not be ignored in the reproduction estimate, and the Committee believes this to be especially a feature of valuation which should depend upon reasonable assumptions based on known occurrences in connection with the property under consideration or similar properties.

A determination should be made in regard to whether the property under consideration is a "losing venture" which cannot earn a fair return on the "fair value" of the property. In such a case, development expense cannot be obtained, because it is based necessarily on the theory of a fair return. In such cases, also, the development expense is unimportant, because the return is independent of the "fair value" of the property deduced either from original cost or reproduction cost.

It is not to be assumed that merely because the property failed to earn an adequate return for a certain number of years, the project was a losing venture. Thus many, if not most, water-works properties built prior to the Nineties, generally required a period of well over a decade to develop normal revenue. Their growth was slow during

the first 15 to 20 years in many cases, nevertheless it cannot be doubted that higher returns could have been collected from their consumers in the majority of cases had it been possible to raise the rates.

Properties vary greatly in respect to the length of time required for developing the business. Some classes of property, like street railways in or between populous cities, may develop their business to a profitable stage in a very short time, but in other cases, as, for instance, a great irrigation project in the Far West, it may be well known in advance of construction that it will take many years to reach the point where profitable results can be achieved. The difference between properties in this respect is obviously to be ascertained by a study of their history and of the history of similar properties.

CHAPTER VIII.

INTANGIBLE VALUE.

In addition to the physical elements of property discussed in the foregoing chapters of this report, there are certain elements of non-physical character that must be considered, and their bearing on the valuation of public utility properties explained. These elements of value are called "intangible" for the reason that they cannot be itemized, measured, and appraised, like physical units, but must generally be estimated by a study of the income account, the possible growth or appreciation in value of property, and other advantages, existing and potential, that attach to the location, environment, ownership, or control of the particular property to which they apply.

Meaning of "Intangible" as Used in Valuation.

The term "intangible" is used in the current literature of taxation to mean bonds, notes, stocks, and money at interest, which, if owned by a corporation whose property is being appraised, must certainly be classed as tangible in the inventory. In the valuation of public utilities, the word "intangible" should be understood as being restricted to its essential meaning of something incorporeal, not capable of direct or exact physical measurement, and in this report its meaning will be thus restricted.

The word "value" has been used very broadly in discussions of valuation and in Court decisions pertaining to the subject. There is a distinctly intangible basis for some of its meanings, and the Committee deems it proper to include the analysis of the word in this chapter.

The chapter then will cover discussions under the following heads:

Value—in general;
Franchise;
Going Value—Going Concern Value;
Good Will;
Efficiency, Favorable Business Arrangements, Design;
Strategic Location;
Leases, Easements, Traffic and Operating Agreements, Water Rights, Strategic Advantages, Other Privileges.

Value, Tangible and Intangible Bases.

Value may be said to rest on both tangible and intangible bases. Some meanings of the word are "excellence", "importance", "utility", "worth". Other meanings are "the desirability of a thing, as com-

pared with the desirability of something else"; "the degree of want felt for a thing, as shown by the relation of supply and demand"; and, commercially, "the price at which a thing will sell". This last is the "exchange value" of this report.

Still other authorized definitions are "the cost to produce"; "the amount of labor necessary to produce". These two definitions furnish the tangible basis for computing the "physical value". The definitions leading up to exchange value furnish the basis for the conception of the intangible element of the term.

We see by this that the conception of the tangible element of the value of a thing rests on its proper cost or on its production, while the exchange value, which includes all tangible and intangible elements of value, rests on the output of the thing or on the results of its operation. Estimates, in money, of these two conceptions may be said to approach the same objective from opposite directions and in an *ideal* public utility, where the actual net returns are just equal to fair income on investment, the figures should be equal. Such equality is obviously a proper aim of regulation. We have then this equation as *ideal* for the *ideal public utility of the future* under constant public regulation from the beginning, but not applicable in general to presently existing properties nor to any property without corresponding contractual obligations—

Tangible value—that is to say, proper cost, including development expense, less "depreciation of valuation".	=	Exchange value—that is to say, dependable net earnings, after deducting a proper depreciation allowance, capitalized at a rate of return, fair for the property in question.

Courts and commissions have striven to establish just valuations of corporate properties for various purposes, such as: a base on which return on capital shall be computed, or on which taxes shall be assessed; a sale made to the public, or against which securities shall be issued. This base has been variously called "fair value", "real value", "reasonable value", "the present value", "actual value", "the value", etc. In the great majority of cases, the properties being considered have been those not under full and continuous regulation in the past; and, where the last member of our equation has figured out larger than the first, the authorities have frequently allowed something for the excess of exchange value over tangible value which has grown up around the nucleus of the latter. This excess, when existent, is called by the Committee the intangible value pertaining to the property. Such additions to tangible value will be fully discussed later in this chapter. The object here is to differentiate clearly between the nature of tangible and intangible values in general, and

to show that both have distinct and legitimate meanings, but that the term "value", standing alone, may lead to misunderstanding of meaning unless qualified by a modifying adjective.

A broad basis for separating tangible from intangible values is that the former can be ascertained from the proper cost while the latter can only be computed from a study of the results which may flow from the use of the properties in question.

Exchange Value.—Exchange value is the desirability of the property, estimated in money, where there is a ready market for properties of the kind considered; exchange value is synonymous with market value, and may be said to be the price which a willing and intelligent buyer will pay a willing and intelligent seller. In the case of a utility property furnishing service to the public, this price will be a function of its existing or potential dependable net earning capacity, or of the additional earning power that may accrue to its owner through the control of the property in question. It is the second member of the above equation. If the property is operating under a limited franchise, the limit will have a bearing on the dependable feature of the earnings or other advantages, and the price will be discounted accordingly. It will be influenced largely by the intangible element in the value of the business as well as by the tangible value of the property measured by cost.

Since exchange value is determined by a consideration of dependable income and beneficial results, existing and potential, it will automatically include all intangible values pertaining to the property, so that the value of franchise, going value (not development expense), goodwill etc., cannot be itemized separately in a schedule.

Fair Value.—The "fair value", of Courts and commissions is intended no doubt to meet the definition given in the Glossary: "A value which is fair to the parties affected". In very many cases where "fair value" has been determined by public authority it has been made up of the present cost value of the property, as nearly as that could be determined, with reasonable allowance for depreciation plus an addition for such intangible features as the controlling authority deemed proper to add to the tangible amount. It has been held time and again by Courts that the "fair value" of a public utility property cannot be determined by a mathematical formula, by which, it is reasonable to infer, is meant a precise estimate based only on an inventory of the physical property.

It will be noted that, in all cases where additions of an intangible nature have been made to the physical inventory, such additions have been in very round numbers; like $285 000 in the Kansas City Water case; $40 000 in the Newburyport Water case; $75 000 in the Gloucester Water case; $150 000 in the Bristol and Warren Water case; $15 000

in the Galena Water case, etc. The roundness of these sums suggests that they were not arrived at by mathematically capitalizing earnings, but by some process of compromise or adjustment which seemed equitable to the Court.

In no recent case before the higher Courts has a valuation been fixed for a public utility property by capitalizing earnings to the exclusion of consideration of its tangible value measured by cost. This procedure seems rational, for the reason that the "fair value" sought by the Courts and commissions is wanted for some purpose of regulation; and, as the object of regulation is to secure justice for both the owners and consumers, it is not conceivable that a value based simply on a consideration of the present or estimated net earnings could be taken as a just base for any of these purposes.

Present earnings may be too low to command capital or to maintain the existing enterprise, or higher than the service rendered may warrant. They are often adventitious, empirical, and based on what the patrons can or will pay. To use them as the sole basis for determining "fair value", would be to render regulation nugatory.

If it is held that fair earnings, for the property as a whole, might be assumed as a starting point, it is pertinent to inquire on what such earnings should be based, or how should they be assumed. It is difficult to use value of service, to the consumer, *per se*, because it cannot generally be computed or assumed, as it is generally intangible and without a basis from which to start. Heat and electric units, energy measured by horse-power and similar units, are constant the world over, and of service to mankind; but no one would claim that their money equivalent is the same everywhere. An inland city of New England, for instance, should pay far more for such units than should a city at the pit-mouth of a coal mine in West Virginia. This illustration means that the cost of service should gauge, to a greater or less extent depending on circumstances, the price or value of it. A substitute plant might furnish service at a less cost, but it has been shown that substitute plants do not generally furnish a safe criterion of value. Hence—in view of the impracticability of assuming fair earnings, and the difficulty, if not uncertainty, attending their computation by other means—the cost of service, as produced by the plant under review, should be accepted as generally the most significant measure of the value of the service, and of the fair earnings to be derived therefrom. The reasonable value for regulation purposes must be largely deduced from the cost basis, with proper consideration of all the attendant circumstances and conditions - as for instance, the value of water rights in regions where water for irrigation has come to have, in fact, a commodity value.

In the opinion of the Committee, *"fair value"*, that is say, the valuation to be assumed in regulation, should be deduced in each case

as it arises, giving full recognition to the tangible values existing in the property, however acquired, and full recognition to the intangible values, so far as they are applicable to the purposes in mind; that *exchange value* should be deduced largely by a consideration of the beneficial results flowing, or reasonably expected to flow, from the use of the property; and that when the word *"value"* is used alone, the text or modifiers should show clearly, and to the extent that may be equitable, whether tangible or intangible value, or the sum of the two, is meant.

Franchise.

This term is defined in the Glossary as:

"The authority granted by government to a person or corporation to create and operate a public service property, to use public property, and often to condemn property."

A franchise of this kind frequently, either explicitly or by implication, grants protection from competition by similar enterprises.

Various Functions.—A franchise is said to be perpetual when it is so granted, or when granted for a very long period; limited, when its term is relatively short or definitely stated; and terminable, when it may be revoked at any time. Terminable franchises are often perpetual in fact, and are sometimes called "indeterminate franchises".

We may divide the possible functions of a franchise into three parts:

1. A license to do business, to operate a public utility, for instance, taking toll from consumers for its output;
2. A grant to use public property, like streets, for subways, pipes, tracks, overhead wires, etc.;
3. A protection from competition.

Generally, in the past, a franchise has been a free grant from the public, and the obvious inference is that the grantee shall perform the service contemplated at a rate to the public such that only a fair return on the capital and effort expended will be secured, including of course proper recognition of the risk involved. If it were supposable that consideration of a franchise would add materially to the capital base, so that rates for service would thereby be enhanced above a fair return, we should be confronted with the anomaly of the public granting a corporation the right of unlimited usury in its dealings with the public.

Court Decisions.—A leading decision of the United States Supreme Court, not concerned with rate regulation, plainly asserts that the franchise has value. In this case (Monongahela Navigation Co. *v.* United States, 148 U. S., 312, March 27, 1893), the Court said:

"So before the property can be taken away from its owners, the whole value must be paid; and that value depends largely upon the productiveness of the property, the franchise to take tolls."

This was a case of the United States taking over from the Canal Company a very old property, and the total award, which was much greater than the cost of reproduction of the physical property, was based on the profitableness of the plant.

In the New York Consolidated Gas case (Willcox *v.* Consolidated Gas Co., 212 U. S., 19, January 4, 1909), relating to rates for gas, the question of the value of franchises was treated at considerable length. This company was created by the consolidation of seven independent companies in 1884, and at that time the State authorities sanctioned the issue of additional capital to represent the value of the franchises as then determined to a sum amounting to $7 781 000. Later, in 1907, the value of the franchises of the company was under consideration, and it was assumed by a master in one case, and by the Federal Court in another, that the franchises had increased in value from the sum above stated to $20 000 000 and $12 000 000, respectively, the increase corresponding to the increase in the value of the physical plant. The United States Supreme Court, in 1909, sanctioned the continuance of the original $7 781 000 of franchise value allowed by the State authorities in 1884, for the reason that its capitalization had been authorized at that time, but it refused to allow any increase of franchise value due to growth of business after that time. The caution as to precedent contained in the second quotation below shows that the Courts are reluctant to recognize the propriety of capitalizing franchise values. The Court said (at pages 44 and 48):

"It cannot be disputed that franchises of this nature are property and cannot be taken or used by others without compensation."

"What has been said herein regarding the value of the franchise in this case, has been necessarily founded upon its own peculiar facts, and a decision thereon can form no precedent in regard to the valuation of franchises generally where the facts are not similar to those in the case before us. We simply accept the sum named as the value under the circumstances stated."

In the case of Kennebec Water District *v.* Waterville (97 Me., 185, December 27, 1902), the Court said (at page 218):

"It [defendant's claim] is that the value of a franchise depends upon its net earning power, present, prospective, developed and capable of development, at reasonable rates, that the value to be assessed is the fair value to the seller and not the buyer, and that 'just compensation' means full compensation for everything or element of value taken. * * * The appraisal must be made having in mind what we have already said concerning the character and duration of the franchise

and reasonableness of rates. While with these limitations, the owner is entitled to receive the value of the franchises, having reference to their prospective use as now developed, and to the future development of their use, consideration must also be had of the fact that further investment may be necessary to develop the use, and of the further fact that at any stage of development the owner of the franchise will be entitled to charge only reasonable rates under the conditions then existing. But, subject to such limitations, we think it should be said that the owner is entitled to any appreciation due to natural causes, such as, for instance, the growth of the cities or towns in which the plant is situated.

And further (at page 220):

"But we cannot assent to the proposition that the capitalization of income even at reasonable rates can be adopted as a sufficient or satisfactory test of present value. Such a capitalization would fix at the present time a specific value which would continue for all time to come, as a fixed and unvarying source of income, no matter how conditions may be changed."

In Spring Valley Water Works *v.* San Francisco (165 Fed., 667, October 7, 1908), the opinion, in discussing the question of franchise value in a rate case, after stating that a franchise, if it has value, should be taken into account in rate regulation as well as condemnation proceedings, said:

"Serious and perplexing questions arise when it is attempted to ascertain the value of the franchise and going business. Here very little has been settled by the Courts, except that each case must depend on its own special circumstances."

In these and other cases the Courts recognize the propriety of including intangible elements of value, but the reasonableness of the rates from which these values are derived is strongly insisted upon. In the case of the Monongahela Navigation Co., a very old property was at bar, and evidently great consideration was granted to long-accepted rates unaffected by regulation, on one hand, and, on the other, to the property rights growing up from these rates on the "innocent owner" idea. In general, the Courts have laid down no rule for fixing reasonableness of rates in these cases, nor for computing the value of a franchise.

Considering the money value of a franchise: as a license to do business, its value is vital, and cannot be measured in money; it is comparable to the value of the air we breathe; as a grant to use public property, it may carry with it costs, rent, fees, or obligations, the cost of which can be estimated. Such costs can hardly be called intangible, and should certainly be included in an inventory. Such a grant may also lessen the expense of installation which would be necessary if streets, etc., could not be used by the utility. This acts

to reduce the necessary investment. The third possible feature of a franchise, protection from competition, is the one that has probably given the principal money value to franchises in the past. It is the legal base for the monopolistic rights of corporations, and is the feature that calls for public regulation more than any other. Intangible values acquired under this phase of franchise have been recognized by the highest Courts to be the inalienable property of the present owners, as shown above, but, unless such values have been converted into physical property useful to the utility, the Courts are now very chary about admitting them to capitalization, as may be seen by the second quotation from the Kennebec Water District case given above. In fact, the principles of fundamental equities would seem to prohibit earning on intangible capital which is admittedly derived from anticipated earnings in excess of a fair return.

In fixing upon the "fair value" for a property, the license feature of a franchise can hardly be included; neither can the privilege to use public property, except so far as such privilege costs the grantee money. In fact, the grant is made purposely to lessen the otherwise necessary capital, and it would certainly be absurd to augment the capital by assuming an intangible value for the grant. The monopolistic feature is then the only base for intangible value pertaining to a franchise; and this cannot be computed separately from that arising from "going value" which will be discussed in the next section.

Going Value—Going Concern Value.

In very many discussions and Court opinions, this component of value has been hopelessly and blindly mixed with development expense, which latter the Committee has set up as a distinct feature. Clearness of understanding will be enhanced if the separation is adhered to.

Development expense is equivalent to expenditure of capital. Going value, on the other hand, is, in the minds of many, apparently, an indefinite something that a live business has, which a plant without business has not. The fact that a business is connected with its customers, house connections made in a water supply system, the breath of life put into the body, as some express it, seems to have an almost mystic significance to many. It has been called the soul of the enterprise, while the physical plant is the material body.

In the work of Henry Floy, entitled "Valuation of Public Utility Properties", 1912, the subject is discussed under four heads, at page 137 and following. These conceptions briefly and clearly set forth the different methods that have been adopted for discussing going value in rational terms and for measuring it in money.

They are:

First.—"A legal and economic, recognized value, usually determined in an approximate way, in addition to, and over and above the

value of the physical plant, resulting from the putting of said plant into actual and useful operation. The value may be based on merely an actual expenditure in dollars, or an estimated value indicating the worth of the service performed, in transforming the 'dead' into a 'live' income producing property."

Second.—"A very frequent use of the term 'going value' is that of 'capitalized losses', namely, the capitalization of losses incurred during the first few years after a corporation begins business."

Third.—"Going value has also been used to mean the value that obtains from capitalizing the present net earnings of a corporation, *i. e.*, the value of a created income."

Fourth.—"Although the trend of modern movement is toward the reduction rather than the creation of non-physical values, going values have been allowed, particularly in water-works appraisals, based on the present worth of estimated earnings including those of future business growth.

"The method consists in estimating the present worth of excess earnings of an existing plant compared with those of a hypothetical plant, between the date of valuation and the time the earnings of the assumed plant shall equal those of the existing plant."

These four interpretations cover the ideas that have been rather vaguely expressed by Courts, commissions, authors, and others, perhaps as well as they can be covered.

It will be seen that the first, second, and fourth are really founded wholly or in part on a conception of what the Committee has classed as development expense; and that the third is the "exchange value" of this report.

A careful reading of Court decisions, attorney's briefs, and opinions of engineers and authors on this subject, leads to the impression that lawyers and judges have generally leaned toward an appreciation of the intangible nature of going value as something inherent in a "system in successful operation and earning a revenue", while engineers and authors generally are disposed to look upon it as the cost of developing, not only the capacity to earn, but the market for the output as well.

Mr. Justice Brewer, National Water Co. *v.* Kansas City (62 Fed., 853, July 2, 1894), said:

"The fact that it is a system in operation, not only with a capacity to supply the city, but actually supplying many buildings in the city —not only with a capacity to earn but actually earning—makes it true that 'the fair and equitable value' is something in excess of the cost of reproduction."

On the other hand, Mr. Benezette Williams, in connection with the Dubuque Water-Works appraisal said, regarding the valuation of the Kansas City Water-Works, after describing the appraisal of the physical property:

"The commissioners, however, took into account the fact that the plant was in actual operation, with an established business and a long list of patrons, and added to the cost of replacement, which included a liberal allowance for contingencies, an allowance for interest on the investment for a time long enough to enable the business to reach the point at which it was found."

These two quotations typify the difference of conception that exists between many students of this subject. The first implies that the judge's conception of the amount to be allowed above reproduction cost was of an intangible nature, presumably to be determined by a study of the income. The other, representing more generally the engineer's view, based this addition on what the Committee considers tangible grounds of interest during construction and development expense. The Committee aims to clarify the subject by assigning all costs like those referred to in Floy's first class, as above, to the appropriate items of the reproduction schedule, and by classing the costs and losses of the development period as development expense and confining going value strictly to its intangible aspect, depending on a study of the income account which reflects the advantages accruing from enlarged business, social growth, and other factors of improvement and progress. Floy's third class does not exactly define the Committee's idea, because it means the whole exchange value of the property, while the Committee considers going value to be a sum that may be, under some circumstances, added to the appraisal of the physical property. It is the difference between the exchange and physical values, and includes and is the resultant of the intangible values pertaining to all the features discussed in this chapter. No division is possible between the value of franchise, favorable location, good-will, or the other elements of advantage. Their effects automatically add together to make up what is distinctly going value.

If, from the actual net earnings for a given year available for distribution there is subtracted a fair return on the reproduction cost, including therein all proper overhead charges, development expenses, and other tangible capital items, including appreciation in value, less "depreciation of valuation", the remainder will be a measure of the income from going value for that year. Whether or not such income may be capitalized and added to reproduction cost is debatable. If the remainder, thus obtained, is not zero, its algebraic sign will determine whether the going value is positive or negative in the case under review.

Going value derived from large earnings is legitimate and inevitable in competitive utilities, like the steam railroads of this country, steamship lines, and most manufacturing enterprises, the favorable location and good management of which give them advantage over rivals less favorably conditioned. In competitive enterprises the

favorably located and well managed one will naturally and properly receive a higher rate of return on its cost than the other one.

When several properties are to be combined in a single corporation, their exchange values seem to furnish the principal basis for adjusting the financial ratios between them.

Good Will.

This feature is defined as follows in the Glossary:

"In ordinary business, the good will represents the special value that a business has by reason of its popularity and the amount of business thereby attracted. It is a part of the value of a 'going concern'."

As applied to a department store or to professional practice, goodwill is a large item of intangible value. In some kinds of competing utilities it undoubtedly has weight. Its effect in any case merges into the resultant described in the last section, and the conditions accruing among competitive utilities are the same with this as with going value. In monopolies the highest Court has decided that good will is not an element to be considered independently in the valuation. In the case of Willcox *v.* Consolidated Gas Co. (212 U. S., 19, January 4, 1909), Mr. Justice Peckham said:

"We are also of the opinion that it is not a case for a valuation of 'good will'. The master combined the franchise value with that of good will, and estimated the total value at $20 000 000.

"The complainant has a monopoly in fact, and a consumer must take gas from it or go without. He will resort to the 'old stand' because he cannot get gas anywhere else. The Court below excluded that item, and we concur in that action."

Efficiency—Favorable Business Arrangements—Design.

These features are important, and, together with good management, should be encouraged. They add largely to the intangible value of a property; being, in fact, a major factor in going value which has been discussed above.

It is often said that efficient, economical management is not encouraged by regulation; but this should not be the necessary result. The English method of gas company regulation called the "London Sliding Scale" is perhaps the most prominent attempt to encourage efficiency that has yet been tried. It was adopted in 1904 by a Legislative act of the State of Massachusetts in an arrangement with the Boston Consolidated Gas Co. At that time the price per thousand feet of gas was fixed at 90 cents, and the dividend to the stockholders of the company at 7%, and the provision made that if in any year the price was reduced below 90 cents the rate of dividend for the succeeding year might be raised one-fifth of 1% for each cent of reduction below 90 cents. Provision was made in the Act for creating a surplus fund, if the earnings were not entirely exhausted by the dividends allowed,

and for increasing the price to consumers, if necessary in the judgment of the Gas and Electric Light Commission. The Act, following English precedent, provides that additional stock may be issued with the approval of the commission, but such issue shall be sold at public sale under competitive bids; also that the standard price, the scale, and the dividend rate shall be subject to readjustment after ten years from the passage of the Act. At present (1916) the price of gas under this arrangement is 80 cents and the dividend rate 9 per cent. The regulating board has radical changes in the method of control under consideration. In England the scheme has been in operation for thirty years or more, and does not work with entire satisfaction, except for a limited number of years after its adoption. In some cases the companies have not increased dividends as the scale allows them to do, evidently on account of the possibility of competition from other producers. This shows that natural competition, left free to act, is the real regulator where competition may exist.

Though the group of factors named in the caption of this section adds greatly to the exchange or market value of a property, they cannot in general be separately included in an inventory of either original or reproduction cost; but where exceptional efficiency in design, construction, and management is manifest, its results, although intangible, should be recognized and encouraged by the regulating body.

Leases—Easements—Water Rights—Traffic and Operating Agreements—Strategic Location and Advantages—and Other Privileges.

These are often elements of substantial value. They are entitled to careful analysis and consideration in accord with their just dues. They are in the nature, however, of special problems, often highly complex and legal in character, varying greatly with locality, environment, and attendant circumstances. Therefore they will not be discussed in detail herein.

Conclusion.

With reference to intangible value as a whole, the Committee takes the ground that, in finding original or reproduction cost, there must be included first, the *tangible* elements (including development expense) to which cost can be assigned or which can be separately scheduled, with an attached value based on concrete facts; and, second, that pertinent facts, bearing upon the *intangible* values, should thereafter be developed independently, as an aid to the formation of sound judgment as to their value.

Exceptional efficiency—although practically difficult of recognition and compensation in any plan of valuation, and outside of strict engineering analysis—should be recognized by Courts and commissions, when appraising property.

GLOSSARY.

Accounting Cost.

See *Book or Accounting Cost,* under COST—INVESTMENT.

Accounting—Depreciation.

See DEPRECIATION ACCOUNTING.

Accounting—Methods of.

See DEPRECIATION ACCOUNTING.

Actual Cost.

See *Original Cost,* under COST—INVESTMENT.

Additions.

Structures, facilities, equipment, and other property units added to those in service and not taking the place of any property for like purposes.

See also BETTERMENTS.

Age.

Under EXPECTATION OF LIFE—LIFE TERM.

Amortization Fund.

See SINKING FUND.

Amortize.

To extinguish a liability or debt by means of a sinking fund. The word, however, is frequently used in connection with such extinguishment by other means.

Annuity.

An annual allowance or payment; generally an equal sum each year, such as the payment to a sinking fund.

Appreciation.

A general definition is gain in value or worth. As used in connection with the valuation of public service property, this term is generally restricted to physical items, and measures their gain in value due to age, use, and properly directed labor. Its principal application in connection with railroads is to roadbed, which, for instance, is increased in value by solidification and grassing of slopes.

The appreciation in land values will be called an *increment.*

Average, Weighted.

See WEIGHTED AVERAGE.

Betterments.

Physical changes in roadway, structures, facilities, or equipment, the object of which is to make the properties affected more useful, efficient, or safe, or of greater capacity.

Book or Accounting Cost.

Under COST—INVESTMENT.

Capital.

"A portion of wealth which is set aside for the production of additional wealth.

"While this is the sense in which the word 'capital' is used in economics, in bookkeeping the term 'capital account' is often used to mean assets on the credit or passive side which denote proprietorship."*

Working Capital.—The capital in the form of cash, stock on hand, and other current assets, which must be kept available for the operation, maintenance and administration of a property.

Capitalization Base.

Under VALUE.

Charges.

See OVERHEAD CHARGES.

Charter.

A grant of power by the proper authority to a group of individuals to become a corporation, in order that they may act as a unit for a specific purpose.

Compound-Interest Method.

See under DEPRECIATION ACCOUNTING.

Construction Period.

That part of the production or reproduction period extending from the inception of an undertaking to the completion of its construction and the beginning of its regular operation. The definition may apply either to a whole property, or to substantial parts of it constructed at different times.

This period is followed by the *physical development period,* in which are incurred construction costs necessary to bring the physical property to its present operating efficiency. (See Reproduction Period.)

Cost.

Cost-Investment.—In the case of a new public service property, in which nothing has been added or abandoned, the outlay incurred in creating the property, not only money outlay, but also the money value of services rendered and of other considerations involved.

In the case of old properties to which many additions have been made, and wherein many items of property have been worn out and replaced one or more times, either in kind or otherwise, these words, when not qualified, have no generally accepted meaning, and have been used by the Courts, commissions, and others in a very loose way.

* From "The Accountancy of Investment", by Charles E. Sprague.

The words are frequently qualified, as, for instance, original cost, actual cost, original cost to date, original cost plus improvements, but even these qualified terms appear to have no generally accepted meaning.

Book or Accounting Cost.—The cost as shown by the books. This necessarily depends on the method of bookkeeping used. It is frequently the case that book cost fails to include the value of services rendered in promotion and construction, for which no money payment was made, and such costs as those due to additions and betterments, interest on capital during construction, and the bringing of the property to its full present condition of operating efficiency, which may have been paid for, out of earnings or otherwise, but properly belong in the capital account. On the other hand, it may include charges for replacements which do not properly belong to the capital account, and may continue to include investment in property which has been retired without being credited on the books.

Cost of Reproduction—Reproduction Cost—Replacement Cost—Cost of Duplication.—The estimated cost of reproducing the property without deduction for the loss of value due to age or other causes.

Original Cost—Original Cost to Date—Original Cost Plus Improvements—Actual Cost.—These terms are often used interchangeably, though suggesting different things. The Committee believes that "original cost to date" and "actual cost" are the better terms and defines them as the cost of the original construction, plus all charges against capital proper, under approved accounting principles, for expenditures incurred thereafter, and minus all proper credits to capital for the cost of property which has been disposed of or otherwise retired.

Under this definition the "original cost to date" or "actual cost" of a property is the first cost of the identical property units now in use, including overhead charges. The former is a term used in the Federal Valuation Act, and may be authoritatively defined by the Interstate Commerce Commission.

Cost of Reproduction.

Under COST—INVESTMENT.

Current Repairs.

See REPAIRS.

Decrement.

See DEPRECIATION.

Decrepitude.

Decretion.

Under DEPRECIATION.

Deferred Maintenance.

A technical term used in connection with public service properties, representing the amount which must be expended in repairs and renewals to bring plant units to an operating condition normal for the character of service to which they are devoted.

Depletion.

Under DEPRECIATION.

Depreciated Value.

See *Remaining or Depreciated Value*, under VALUE.

Depreciation.

A general definition is lowering in value or worth. As used in connection with the valuation of public service property, it is the loss of value due to age and use, including the loss from deterioration, wear and tear, inadequacy, obsolescence, depletion, and other similar causes.

Discussion of depreciation generally includes, also, causes of loss of value or worth, and methods of depreciation accounting and of restoring the loss to the property or its owners.

The depreciation in land values will be called a *decrement*.

Accrued Depreciation.—Depreciation which has taken place. It may have reference to existing property only, or it may include also the abandoned property.

Decrepitude.—A term sometimes used in connection with public service properties as having a meaning similar to deterioration.

Decretion.—Loss of service life.

Deferred Maintenance.—See DEFERRED MAINTENANCE.

Depletion.—Permanent loss of value due to exhaustion, or permanent lessening of value, as, for example, of mining properties.

Depreciation of Valuation or Fair Depreciation.—The sum which should be deducted from original cost to date or from estimated cost of reproduction new, as a step in finding that which the courts have called "fair value".

Deterioration.—Reduction in the quality of a property unit or in its efficiency for service, due to its physical condition.

Fair Depreciation.—See DEPRECIATION OF VALUATION, *supra*.

Functional Depreciation.—Depreciation due to inadequacy, obsolescence, and supersession.

Inadequacy.—The insufficiency of a unit to perform fully the function required of it.

Obsolescence.—The condition or process by which units gradually cease to be useful or profitable as a part of the property, on account of changed conditions.

Physical Depreciation.—Depreciation due to deterioration, such as use, wear and tear, accident.

Supersession.—The substitution of a more profitable or efficient unit for the existing property unit.

Wear and Tear.—The physical deterioration of a property unit occasioned by its use.

Depreciation Accounting.

The branch of accounting by which loss of value is provided for or accounted for either wholly or in part. There are various methods, among which are:

1.—Replacement Method.—This method makes provision from the earnings for reimbursement for the cost, less salvage, of property units retired when they reach the ends of their service lives, whether for replacement or for any other reasons.

2.—Sinking-Fund Method.—This method makes provision from the earnings for equal annual depreciation allowances of such size for any property unit that if these allowances are invested in a sinking fund they will, *with accumulations of interest compounded annually,* amount to the cost, less salvage, of the unit at the time when it reaches the end of its estimated service life. In applying this method, it has been common practice to compute annuities upon a basis of original or reproduction cost and total service life, but the method applies as well if, at any time, actual value and remaining service life are used.

3.—Compound-Interest Method.—(The same as the *Equal-Annual-Payment Method* discussed in the Committee's Progress Report.) This method makes provision from the earnings for annual depreciation allowances, for any property unit, of such increasing sizes from the beginning to the end of its estimated service life that when the same interest rate is used for determining the return on capital and the depreciation allowances, the sum of the return and the allowance is the same in each year of such life. The rate of increase of the successive depreciation allowances is also such that the "present worth" of each is the same. The sum of the allowances *without interest* equals the cost, less salvage, of each unit when it reaches the end of its estimated service life.

4.—Straight-Line Method.—This method makes provision from the earnings for equal annual depreciation allowances of such size, for any property unit, that the sum of the allowances, *without interest,* will equal the cost, less salvage, of each unit when it reaches the end of its estimated service life. The annual depreciation allowances by this method are each equal to the cost, less salvage, of the property unit divided by the number of years of its expectation of life.

5.—Unit-Cost Method.—This method is based on the conception that the value of a plant unit should be decreased from year to year to such an extent that the cost per unit of output or service, taking into account all annual charges for interest, depreciation, repairs, cost

of operating, etc., shall be constant during each year of the estimated service life of the unit.

Depreciation Allowance—Renewal Allowance—Retiring Allowance.

The provision which should be made through the rates by which a sum may be taken from earnings periodically for offsetting the decrease in value or worth of existing plant units. In accordance with methods of accounting in general use by railroads for a part of their property, and by some other public service corporations, the annual decrease in value or worth is offset wholly or in part by charges to maintenance or renewal expenses.

Depreciation Fund—Renewal Fund.

A fund consisting of cash or other quick assets set aside for the purpose of renewing property subject to depreciation when it ceases to be serviceable. This fund appears in the asset column of a financial statement, and may have no direct relation to the depreciation reserve.

Depreciation Reserve—Retiring Reserve.

The excess of past charges to "operating expenses" for depreciation which have been credited to the depreciation reserve account, over and above those retirements which have been charged thereto.

Deterioration.

Under DEPRECIATION.

Development Expense.

A term which may be applied to all expenses incurred in developing a property. In this report, however, it is limited to the expenses incurred in developing the property and business after the period of regular operation begins. That portion of development expense incurred in tuning up the plant and bringing the property to present operating efficiency during the physical development period may be accounted for as a construction cost. To a large extent, development expense represents the deficiency in net distributable earnings below a "fair return" on the "fair value" of the property during the early period of operation after original construction, and after important betterment or reconstruction. In other words, it is the cost of creating the business.

Development Period—Physical.

See PHYSICAL DEVELOPMENT PERIOD.

Duplication Cost.

See *Cost of Reproduction*, under COST—INVESTMENT.

Equal-Annual-Payment Method.

Under DEPRECIATION ACCOUNTING.

Exchange Value.

Under VALUE.

Expectation of Life—Life Term.

These terms mean the estimated economically useful life of the different units or different groups of units of which a property is composed.

Age.—The length of time that the property unit has been in use. Sometimes referred to as past life.

Remaining Life—Remaining Service Life.—Generally, the difference between the age and the total expectation of life of a property unit; but, as the unit grows older, the estimate of remaining service life, sometimes termed future life, may be changed by the condition of the property at the time of its appraisal.

Expense—Development.

See DEVELOPMENT EXPENSE.

Expenses—Operating.

See OPERATING EXPENSES.

Fair Return.

This term is used to indicate the return to which the owner of a public utility is equitably entitled for the use of his capital, in view of the risks involved and the efficiency displayed in management.

Fair Value.

Under VALUE.

Franchise.

The authority granted by government to a person or corporation to create and operate a public service property, to use public property, and often to condemn property.

Going Value—Going Concern Value.

These terms have been given many definitions. Mr. Justice Moody, in the Knoxville case, referring to an allowance for the going concern, makes this statement:

"The latter sum, we understand to be an expression of the added value of the plant as a whole, over the sum of the values of its component parts, which is attached to it because it is in active and successful operation and earning a revenue."

The term "going value" has also been given a less comprehensive meaning, in the valuation of public service properties, making it correspond closely with what has been defined heretofore as "development expense"; for instance, Judge Miller, in the Kings County Lighting case, said:

"I define 'going value' for rate purposes as involved in this case to be the amount equal to the deficiency of net earnings below a fair

return on the actual investment due solely to the time and expenditure reasonably necessary and proper to the development of the business and property to its present stage, and not comprised in the valuation of the physical property."

The Committee defines the term as the difference between the exchange value and the physical value of an established and operating property, due to the *intangible* values—such as franchise, good will, efficiency, favorable business arrangements, designs, strategic location, leases, easements, traffic and operating agreements, water rights, strategic advantages, and other rights or privileges—in contradistinction and addition to the development expense (or *tangible* costs and losses of the business-acquisition, or development-period, which are to be included in the physical value of the property).

Good Will.

In ordinary business, the good will represents the special value that a business has by reason of its popularity and the amount of business thereby attracted. It is a part of the value of a "going concern".

Improvements.

Additions and betterments to an existing property.

Inadequacy.

Under DEPRECIATION.

Increment.

See APPRECIATION.

Intangible Value.

Under VALUE.

Inventory.

The schedule with items multiplied by appropriate prices.

Investment.

See COST—INVESTMENT.

Item.

Under UNIT.

Items—Measurable.

See *Measurable Items,* under UNIT.

Life—Remaining.

See *Remaining Life,* under EXPECTATION OF LIFE.

Life Term.

See EXPECTATION OF LIFE.

Maintenance.

The act of keeping a property in condition to perform adequately and efficiently the service for which it is used. The term may be applied either to the property units which are kept in such serviceable

condition by repairs or otherwise, or to the property as a whole which may be kept in such condition by the replacement of parts which become worn or otherwise inefficient.

Under the rules prescribed by the Interstate Commerce Commission for steam roads, the word "maintenance" has a broader significance, as maintenance expenses include:

"also the loss through depreciation of the property used in operations, including all such expenses resulting from ordinary wear and tear of service, exposure to the elements, inadequacy, obsolescence, or other depreciation, or from accident, fire, flood, or other casualty."

Maintenance—Deferred.

See DEFERRED MAINTENANCE.

Market Value.

Under VALUE.

Measurable Items.

Under UNIT.

Obsolescence.

Under DEPRECIATION.

Operating Expenses.

Strictly speaking, these are the expenses incurred in operating a property, for which its owner is entitled to receive reimbursement through sufficiently large earnings. In accounting, the term has been given a broader meaning to include all expenses necessary for the maintenance of the integrity of the property and its operation.

Original Cost—Original Cost to Date—Original Cost plus Improvements.

Under COST—INVESTMENT.

Overhead Charges.

This is a term used for expenses incurred in producing a property, which cannot well be included in the unit prices. The methods of accounting used in connection with the construction of public or public service works do not, as a rule, provide for the charging of such expenses as those for promotion, administration, engineering, interest, and many other things to the different property units, although they are necessarily a part of the cost of such units. In the preparation of estimates of cost provision must be made, also, for contingencies which experience has shown do arise during construction. Hence, the engineer, using all available accounts and records as a basis, can make the most accurate estimate of reproduction cost by applying unit prices which do not include these overhead charges, and adding for the different elements which make up the overhead charges such percentages of the total cost of the plant units or measurable items of property, otherwise obtained, as are warranted by experience.

Physical Development Period.

The period, following the construction period, necessary to bring any physical property to present operating efficiency.

Physical Value.

Under VALUE.

Plant Unit.

See UNIT.

Prices.

See UNIT PRICES.

Property.

See PUBLIC SERVICE PROPERTY.

Property Unit.

See UNIT.

Public Purchase Base.

Under VALUE.

Public Service Commission.

The general title given to a public body authorized by law to regulate or control the actions of individuals, corporations, or public bodies owning or operating public utilities. As a rule, the authority is conferred on special bodies, as, for example, the Interstate Commerce Commission, and the public service commissions which have been created in recent years in most of the States, but this authority may also be conferred on the governing bodies of cities, as, for instance, in California, where, by the provisions of the Constitution of the State, the supervisors of a city were until recently required each year to fix the rates for water supplied by public service corporations to such city and its inhabitants.

Public Service Property—Public Utility.

The popular names given to a property created, under the provisions of a general or special law, to furnish service to the public. The public utility may be owned or controlled by individuals, a corporation, or a public body.

Public Utility.

See PUBLIC SERVICE PROPERTY.

Rating Base.

See *Return Base* under VALUE.

Remaining Life—Remaining Service Life.

Under EXPECTATION OF LIFE.

Remaining or Depreciated Value.

Under VALUE.

Renewal—Renewal in Kind—Replacement.

Renewal.—The making new of plant units by replacing them with corresponding new units, although the term is sometimes used even when the new units are not altogether similar to those replaced.

Renewal in Kind.—Used to indicate that the renewal is made with an identical plant unit.

Replacement.—The act of replacing a plant unit which is going out of service, with a substitute which may be either identical with the unit replaced or different from it. A replacement which increases the value is classed as an improvement to the extent of its increased cost.

Renewal Allowance.

See DEPRECIATION ALLOWANCE.

Renewal Fund.

See DEPRECIATION FUND.

Renewal in Kind.

Under RENEWAL.

Repairs.

The process of mending or of restoring to a sound and good state. In connection with the valuation of a public service property, the word may have two very different meanings. It may mean either the mending and restoring of parts of a unit to keep it in a sound and good state, or the mending and restoring of the defective parts of the whole plant to maintain it in a sound and good state. The meaning intended is sometimes indicated by the context, and where not so indicated the difference in meaning is a source of misunderstanding.

The frequently used term *current repairs* refers to the repairs belonging to the time immediately passing, and, like the word repairs, may relate either to plant units or to the whole plant.

Replacement.

Under RENEWAL.

Replacement Cost.

See *Cost of Reproduction,* under COST—INVESTMENT.

Replacement Method.

Under DEPRECIATION ACCOUNTING.

Reproduction Cost.

See *Cost of Reproduction,* under COST—INVESTMENT.

Reproduction Period.

The period estimated as necessary for the inception, construction and bringing to present operating efficiency of any property. It em-

braces a construction and a physical development period and the period necessary to acquire the business of the utility.

Retiring Allowance.

See DEPRECIATION ALLOWANCE.

Retiring Reserve.

See DEPRECIATION RESERVE.

Return Base.

Under VALUE.

Salvage or Scrap Value.

Under VALUE.

Schedule.

A detailed list of all of the items of property.

Scrap Value.

See *Salvage or Scrap Value,* under VALUE.

Service Life.

See EXPECTATION OF LIFE.

Sinking Fund—Amortization Fund.

A fund instituted and invested in such wise that its gradual accumulations will enable it to meet and wipe out a liability or debt at maturity. The term "sinking fund" is sometimes used loosely in connection with any kind of a fund which will reach the amount of a liability or debt at maturity, but should be used only in connection with a fund made up of annual or other instalments *which require the accumulation of interest* in order to reach the full value at maturity.

Sinking-Fund Method.

Under DEPRECIATION ACCOUNTING.

Straight-Line Method.

Under DEPRECIATION ACCOUNTING.

Supersession.

Under DEPRECIATION.

Taxing Base.

Under VALUE.

Unit.

A subdivision of a plant or property consisting of a single article or item, or a group of articles or items. The subdivision of the plant or property into units, not necessarily the same for different purposes, for convenience in estimating cost or in estimating and placing depreciation. A *plant unit* is a subdivision of the *physical property*; a *property unit* a subdivision of either *physical* or *intangible* property.

The term *plant unit* generally relates to a complete unit like a locomotive, a building, or a dam, and not to the parts like the wheels of a locomotive, the windows of a building, or the earthwork or masonry of a dam.

The word unit may be at times synonymous with the word item, but frequently will include two or more similar or dissimilar items. For instance, a dam may be called a property unit, but, in estimating its cost of reproduction, its various parts, such as earthwork, masonry, paving, etc., would be entered as items in the schedule. On the other hand, an item in a summarized schedule often includes many units.

Item.—An article; also a separate article or entry in an account or schedule.

Measurable Items.—Items of physical property which can be counted or measured.

Unit-Cost Method.

Under DEPRECIATION ACCOUNTING.

Unit Prices.

These are the prices or costs to be set opposite each plant unit of the schedule, as multipliers.

Valuation.

The general definition is the act of valuing. In connection with public service property, the word means the determination of actual or reproduction cost, depreciation, and the many other factors which affect "fair value" or the bases for estimating "fair return", permissible capital, public purchase price, or proper taxation and the determination of such "fair value" or bases.

Value.

Primarily defined in the Standard Dictionary as "the desirability or worth of a thing as compared with the desirability of something else". As the desirability or worth of a thing is usually estimated in money, this definition in the present case may be abbreviated to "the desirability or worth of a thing, estimated in money".

As indicated in the introductory chapter, the Committee is of the opinion that the confusion which has resulted from the use of the word, value, because of the many different shades of meaning that have been given to the word, is so great as to justify a modified terminology. It therefore recommends:

(*a*) that the meaning to be attached to the word value be the primary meaning as defined in the first paragraph, except when it is necessary to use the words "value" and "fair value" with the meanings assigned to them by the Courts;

(*b*) that, in so far as the practice of valuation is concerned, the word should not be used, except as above stated, without proper modifying words to indicate what meaning is intended;
(*c*) that the principal purposes of valuation being to furnish a basis for a purchase or rental price, to obtain evidence as a guide for rate-making or the determination of the reasonableness of rates, to furnish a basis for authorization of capital, and to furnish a basis for taxation—these bases not being necessarily the same and not being the equivalent of value in every case—the use of the word value should be avoided and the propriety of the following terms recognized:

Capitalization Base.—The sum on which capital securities may be issued or authorized.

Public Purchase Base.—The price which should be paid in the case of the purchase of a public utility by the public.

Return Base.—The amount on which the public service company is entitled to earn a "fair return". This is what the Courts have termed the "fair value", in various rate cases.

Taxing Base.—The amount which should be subjected to taxation. Under the laws of some States, the basis to be used for taxation of property is not its value estimated in money by any ordinary rules, but a sum determined in accordance with such laws.

Exchange Value.—The desirability or worth of a thing, estimated in money. (See also VALUE and MARKET VALUE.)

Fair Value.—As applied to a public service property, this is a technical term used by the Courts to express a base for rate regulation, public purchase, capitalization, or other purpose, which is fairly obtained and fair to the parties affected.

Going Value.—See GOING VALUE—GOING CONCERN VALUE.

Intangible Value.—Existing value which is not represented by physical property and is not measurable by determinable cost. Intangible value exists by reason of franchise, grants, contract rights, location, market, age, strategic advantages, and efficiency, and possibly other elements that give a property value.

Market Value.—This term, applicable only to a property for which there is a well-established market, has been properly defined as the sum which a willing and intelligent buyer desiring such property would give for it to a willing, intelligent and solvent seller.

Physical Value.—The value of the physical property in contradistinction to that of the intangible property. Riggs,* referring to the "physical property" element of value, says:

"This consists of those things which are visible and tangible, capable of being inventoried, their cost of reproduction determined, their de-

* "The Valuation of Public Service Corporation Property", by Henry Earle Riggs, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXII, June, 1911, p. 18.

preciation measured, and without which the property would be unable to produce the commodity on the sale of which income depends. This physical property is considered as an operating entity, and not as a collection of inert and partly worn-out equipment, and, being so considered, carries, as part of the physical value, those costs and charges which are an inseparable part of the cost of construction but do not appear in the inventory of the completed property."

This definition of physical property is broader than that sometimes given, which excludes the overhead charges. The Committee agrees with this definition of Professor Riggs, as all costs required to create the physical property and put it in its full present condition of operating efficiency should be included in determining the physical value.

Remaining or Depreciated Value.—The value of a property after proper deduction has been made for depreciation.

Salvage or Scrap Value.—These terms are often used interchangeably, but a distinction may be made that *salvage* is the net value which an article possesses for further use for utility purposes other than that for which it was originally used, or as a second-hand article; and *scrap* is the net value of an article no longer useful for service.

Wear and Tear.

Under DEPRECIATION.

Weighted Average.

The average found by dividing the sum of the several products of a series of multiplications of variable factors, by the sum of the factors of one class.

Working Capital.

Under CAPITAL.

APPENDICES TO THE REPORT OF THE SPECIAL COMMITTEE TO FORMULATE PRINCIPLES AND METHODS FOR THE VALUATION OF RAILROAD PROPERTY AND OTHER PUBLIC UTILITIES

Appendix I.....Depreciation Tables.

Appendix II.....Some Examples of Expectation of Life of So-Called Permanent Structures.

Appendix III.....Some Examples of Actual Overhead Cost.

Appendix IV.....Some Records of Abandoned Property.

APPENDIX I.

Accompanying Final Report of the Special Committee on Valuation of Public Utilities.

DEPRECIATION TABLES,

Based on Compound-Interest Method,
Heretofore Called Equal-Annual-Payment Method.

Interest Compounded Annually.

("Value" is value at end of year; "Dep." is depreciation during year.)

5-Year Life.

Age, in years.	Interest Rate. 4%		Interest Rate. 5%		Interest Rate. 6%		Interest Rate. 7%	
	Value.	Dep.	Value.	Dep.	Value.	Dep.	Value.	Dep.
0	100.0000	18.4627	100.0000	18.0975	100.0000	17.7396	100.0000	17.3891
1	81.5373	19.2012	81.9025	19.0023	82.2604	18.8041	82.6109	18.6063
2	62.3361	19.9693	62.9002	19.9525	63.4563	19.9322	64.0046	19.9087
3	42.3668	20.7680	42.9477	20.9501	43.5241	21.1282	44.0959	21.3024
4	21.5988	21.5988	21.9976	21.9976	22.3959	22.3959	22.7935	22.7935
5	0.0000		0.0000		0.0000		0.0000	
		100.0000		100.0000		100.0000		100.0000

10-Year Life.

0	100.0000	8.3291	100.0000	7.9505	100.0000	7.5868	100.0000	7.2377
1	91.6709	8.6623	92.0495	8.3480	92.4132	8.0420	92.7623	7.7444
2	83.0086	9.0088	83.7015	8.7654	84.3712	8.5245	85.0179	8.2865
3	73.9998	9.3690	74.9361	9.2037	75.8467	9.0360	76.7314	8.8666
4	64.6308	9.7439	65.7324	9.6638	66.8107	9.5782	67.8648	9.4872
5	54.8869	10.1336	56.0686	10.1470	57.2325	10.1528	58.3776	10.1513
6	44.7533	10.5389	45.9216	10.6544	47.0797	10.7620	48.2263	10.8619
7	34.2144	10.9606	35.2672	11.1871	36.3177	11.4078	37.3644	11.6223
8	23.2538	11.3989	24.0801	11.7464	24.9099	12.0922	25.7421	12.4358
9	11.8549	11.8549	12.3337	12.3337	12.8177	12.8177	13.3063	13.3063
10	0.0000		0.0000		0.0000		0.0000	
		100.0000		100.0000		100.0000		100.0000

15-Year Life.

0	100.0000	4.9941	100.0000	4.6342	100.0000	4.2963	100.0000	3.9795
1	95.0059	5.1939	95.3658	4.8860	95.7037	4.5540	96.0205	4.2580
2	89.8120	5.4016	90.4998	5.1092	91.1497	4.8273	91.7625	4.5561
3	84.4104	5.6177	85.3906	5.3646	86.3224	5.1170	87.2064	4.8750
4	78.7927	5.8424	80.0260	5.6330	81.2054	5.4239	82.3314	5.2162
5	72.9503	6.0760	74.3930	5.9146	75.7815	5.7493	77.1152	5.5814
6	66.8743	6.3192	68.4784	6.2108	70.0322	6.0944	71.5338	5.9722

15-YEAR LIFE.—(*Continued.*)

Age, in years.	Interest Rate. 4%		Interest Rate. 5%		Interest Rate. 6%		Interest Rate. 7%	
	Value.	Dep.	Value.	Dep.	Value.	Dep.	Value.	Dep.
7	60.5551	6.5719	62.2681	6.5209	63.9878	6.4601	65.5616	6.3901
8	53.9832	6.8348	55.7472	6.8468	57.4777	6.8476	59.1715	6.8375
9	47.1484	7.1081	48.9004	7.1892	50.6301	7.2584	52.3340	7.3160
10	40.0403	7.3925	41.7112	7.5487	43.3717	7.6941	45.0180	7.8282
11	32.6478	7.6882	34.1625	7.9261	35.6776	8.1555	37.1898	8.3762
12	24.9596	7.9957	26.2364	8.3224	27.5221	8.6450	28.8136	8.9625
13	16.9639	8.3156	17.9140	8.7385	18.8771	9.1636	19.8511	9.5899
14	8.6483	8.6483	9.1755	9.1755	9.7135	9.7135	10.2612	10.2612
15	0.0000		0.0000		0.0000		0.0000	
		100.0000		100.0000		100.0000		100.0000

20-YEAR LIFE.

0	100.0000	3.3582	100.0000	3.0243	100.0000	2.7185	100.0000	2.4393
1	96.6418	3.4925	96.9757	3.1755	97.2815	2.8815	97.5607	2.6100
2	93.1493	3.6322	93.8002	3.3342	94.4000	3.0545	94.9507	2.7928
3	89.5171	3.7775	90.4660	3.5010	91.3455	3.2377	92.1579	2.9882
4	85.7396	3.9286	86.9650	3.6760	88.1078	3.4320	89.1697	3.1974
5	81.8110	4.0857	83.2890	3.8598	84.6758	3.6379	85.9723	3.4213
6	77.7253	4.2492	79.4292	4.0528	81.0379	3.8561	82.5510	3.6607
7	73.4761	4.4191	75.3764	4.2554	77.1818	4.0876	78.8903	3.9169
8	69.0570	4.5959	71.1210	4.4682	73.0942	4.3328	74.9734	4.1912
9	64.4611	4.7797	66.6528	4.6916	68.7614	4.5928	70.7822	4.4845
10	59.6814	4.9709	61.9612	4.9262	64.1686	4.8684	66.2977	4.7985
11	54.7105	5.1698	57.0350	5.1725	59.3002	5.1604	61.4992	5.1343
12	49.5407	5.3766	51.8625	5.4311	54.1398	5.4701	56.3649	5.4938
13	44.1641	5.5916	46.4314	5.7027	48.6697	5.7982	50.8711	5.8783
14	38.5725	5.8152	40.7287	5.9878	42.8715	6.1462	44.9928	6.2898
15	32.7573	6.0479	34.7409	6.2872	36.7253	6.5149	38.7030	6.7301
16	26.7094	6.2898	28.4537	6.6016	30.2104	6.9059	31.9729	7.2012
17	20.4196	6.5414	21.8521	6.9317	23.3045	7.3202	24.7717	7.7052
18	13.8782	6.8031	14.9204	7.2783	15.9843	7.7593	17.0665	8.2446
19	7.0751	7.0751	7.6421	7.6421	8.2250	8.2250	8.8219	8.8219
20	0.0000		0.0000		0.0000		0.0000	
		100.0000		100.0000		100.0000		100.0000

25-Year Life.

Age, in years.	Interest Rate. 4%		Interest Rate. 5%		Interest Rate. 6%		Interest Rate. 7%	
	Value.	Dep.	Value.	Dep.	Value.	Dep.	Value.	Dep.
0	100.0000	2.4012	100.0000	2.0952	100.0000	1.8227	100.0000	1.5811
1	97.5988	2.4972	97.9048	2.2001	98.1773	1.9330	98.4189	1.6917
2	95.1016	2.5972	95.7047	2.3100	96.2453	2.0480	96.7272	1.8101
3	92.5044	2.7010	93.3947	2.4254	94.1973	2.1708	94.9171	1.9369
4	89.8034	2.8091	90.9693	2.5468	92.0265	2.3011	92.9802	2.0724
5	86.9943	2.9214	88.4225	2.6742	89.7254	2.4391	90.9078	2.2175
6	84.0729	3.0383	85.7483	2.8078	87.2863	2.5855	88.6903	2.3728
7	81.0346	3.1598	82.9405	2.9482	84.7008	2.7406	86.3175	2.5388
8	77.8748	3.2862	79.9923	3.0957	81.9602	2.9051	83.7787	2.7165
9	74.5886	3.4176	76.8966	3.2504	79.0551	3.0794	81.0622	2.9067
10	71.1710	3.5544	73.6462	3.4129	75.9757	3.2641	78.1555	3.1102
11	67.6166	3.6965	70.2333	3.5836	72.7116	3.4600	75.0453	3.3279
12	63.9201	3.8444	66.6497	3.7627	69.2516	3.6675	71.7174	3.5608
13	60.0757	3.9982	62.8870	3.9509	65.5841	3.8877	68.1566	3.8101
14	56.0775	4.1581	58.9361	4.1485	61.6964	4.1208	64.3465	4.0768
15	51.9194	4.3244	54.7876	4.3559	57.5756	4.3682	60.2697	4.3622
16	47.5950	4.4974	50.4317	4.5736	53.2074	4.6302	55.9075	4.6675
17	43.0976	4.6773	45.8581	4.8024	48.5772	4.9081	51.2400	4.9942
18	38.4203	4.8643	41.0557	5.0424	43.6691	5.2025	46.2458	5.3439
19	33.5560	5.0590	36.0133	5.2946	38.4666	5.5147	40.9019	5.7179
20	28.4970	5.2613	30.7187	5.5593	32.9519	5.8455	35.1840	6.1181
21	23.2357	5.4718	25.1594	5.8373	27.1064	6.1963	29.0659	6.5465
22	17.7639	5.6906	19.3221	6.1291	20.9101	6.5681	22.5194	7.0047
23	12.0733	5.9183	13.1930	6.4356	14.3420	6.9621	15.5147	7.4950
24	6.1550	6.1550	6.7574	6.7574	7.3799	7.3799	8.0197	8.0197
25	0.0000		0.0000		0.0000		0.0000	
		100.0000		100.0000		100.0000		100.0000

30-Year Life.

0	100.0000	1.7880	100.0000	1.5051	100.0000	1.2649	100.0000	1.0586
1	98.2170	1.8543	98.4949	1.5804	98.7351	1.3408	98.9414	1.1328
2	96.3627	1.9285	96.9145	1.6595	97.3943	1.4212	97.8086	1.2120
3	94.4342	2.0057	95.2550	1.7423	95.9731	1.5065	96.5966	1.2969

30-YEAR LIFE.—(*Continued.*)

Age, in years.	Interest Rate. 4%		Interest Rate. 5%		Interest Rate. 6%		Interest Rate. 7%	
	Value.	Dep.	Value.	Dep.	Value.	Dep.	Value.	Dep.
4	92.4285	2.0859	93.5127	1.8296	94.4666	1.5969	95.2997	1.3877
5	90.3426	2.1698	91.6831	1.9209	92.8697	1.6927	93.9120	1.4848
6	88.1728	2.2560	89.7622	2.0171	91.1770	1.7943	92.4272	1.5887
7	85.9173	2.3463	87.7451	2.1179	89.3827	1.9019	90.8385	1.6999
8	83.5710	2.4402	85.6272	2.2288	87.4808	2.0161	89.1386	1.8190
9	81.1308	2.5378	83.4084	2.3349	85.4647	2.1370	87.3196	1.9462
10	78.5930	2.6393	81.0685	2.4518	83.3277	2.2652	85.3734	2.0825
11	75.9537	2.7449	78.6167	2.5748	81.0625	2.4011	83.2909	2.2283
12	73.2088	2.8545	76.0424	2.7080	78.6614	2.5452	81.0626	2.3843
13	70.3543	2.9689	73.3394	2.8382	76.1162	2.6980	78.6783	2.5511
14	67.3854	3.0877	70.5012	2.9800	73.4182	2.8597	76.1272	2.7297
15	64.2977	3.2111	67.5212	3.1291	70.5585	3.0314	73.3975	2.9209
16	61.0866	3.3395	64.3921	3.2856	67.5271	3.2133	70.4766	3.1254
17	57.7471	3.4731	61.1065	3.4498	64.3188	3.4061	67.3512	3.3439
18	54.2740	3.6121	57.6567	3.6223	60.9077	3.6104	64.0073	3.5782
19	50.6619	3.7565	54.0344	3.8034	57.2973	3.8271	60.4291	3.8286
20	46.9054	3.9068	50.2310	3.9936	53.4702	4.0566	56.6005	4.0966
21	42.9986	4.0631	46.2374	4.1933	49.4136	4.3001	52.5039	4.3833
22	38.9355	4.2256	42.0441	4.4029	45.1135	4.5581	48.1206	4.6902
23	34.7099	4.3946	37.6412	4.6231	40.5554	4.8316	43.4304	5.0186
24	30.3153	4.5704	33.0181	4.8543	35.7238	5.1214	38.4118	5.3698
25	25.7449	4.7532	28.1638	5.0968	30.6024	5.4288	33.0420	5.7457
26	20.9917	4.9433	23.0670	5.3519	25.1736	5.7544	27.2963	6.1479
27	16.0484	5.1411	17.7151	5.6194	19.4192	6.0998	21.1484	6.5782
28	10.9073	5.3467	12.0957	5.9005	13.3194	6.4657	14.5702	7.0387
29	5.5606	5.5606	6.1952	6.1952	6.8537	6.8537	7.5315	7.5315
30	0.0000		0.0000		0.0000		0.0000	
		100.0000		100.0000		100.0000		100.0000

35-YEAR LIFE.

0	100.0000	1.3577	100.0000	1.1072	100.0000	0.8974	100.0000	0.7234
1	98.6423	1.4121	98.8928	1.1625	99.1026	0.9512	99.2766	0.7740
2	97.2302	1.4685	97.7303	1.2207	98.1514	1.0083	98.5026	0.8282
3	95.7617	1.5273	96.5096	1.2816	97.1431	1.0688	97.6744	0.8862

35-YEAR LIFE.—(*Continued.*)

Age, in years.	Interest Rate. 4%		Interest Rate. 5%		Interest Rate. 6%		Interest Rate. 7%	
	Value.	Dep.	Value.	Dep.	Value.	Dep.	Value.	Dep.
4	94.2344	1.5884	95.2280	1.3458	96.0743	1.1329	96.7882	0.9483
5	92.6460	1.6519	93.8822	1.4131	94.9414	1.2010	95.8399	1.0146
6	90.9941	1.7179	92.4691	1.4837	93.7404	1.2729	94.8253	1.0856
7	89.2762	1.7867	90.9854	1.5579	92.4675	1.3493	93.7397	1.1616
8	87.4895	1.8581	89.4275	1.6358	91.1182	1.4303	92.5781	1.2429
9	85.6314	1.9325	87.7917	1.7176	89.6879	1.5162	91.3352	1.3300
10	83.6989	2.0098	86.0741	1.8035	88.1717	1.6071	90.0052	1.4230
11	81.6891	2.0902	84.2706	1.8936	86.5646	1.7034	88.5822	1.5226
12	79.5989	2.1737	82.3770	1.9883	84.8612	1.8058	87.0596	1.6293
13	77.4252	2.2607	80.3887	2.0877	83.0554	1.9140	85.4303	1.7432
14	75.1645	2.3512	78.3010	2.1922	81.1414	2.0289	83.6871	1.8653
15	72.8133	2.4452	76.1088	2.3017	79.1125	2.1506	81.8218	1.9959
16	70.3681	2.5430	73.8071	2.4168	76.9619	2.2797	79.8259	2.1356
17	67.8251	2.6447	71.3903	2.5377	74.6822	2.4165	77.6903	2.2851
18	65.1804	2.7505	68.8526	2.6645	72.2657	2.5614	75.4052	2.4450
19	62.4299	2.8605	66.1881	2.7978	69.7043	2.7152	72.9602	2.6162
20	59.5694	2.9750	63.3903	2.9376	66.9891	2.8780	70.3440	2.7993
21	56.5944	3.0939	60.4527	3.0845	64.1111	3.0507	67.5447	2.9953
22	53.5005	3.2178	57.3682	3.2388	61.0604	3.2338	64.5494	3.2049
23	50.2827	3.3464	54.1294	3.4007	57.8266	3.4278	61.3445	3.4293
24	46.9363	3.4803	50.7287	3.5708	54.3988	3.6334	57.9152	3.6693
25	43.4560	3.6195	47.1579	3.7492	50.7654	3.8515	54.2459	3.9262
26	39.8365	3.7642	43.4087	3.9367	46.9139	4.0826	50.3197	4.2010
27	36.0723	3.9149	39.4720	4.1337	42.8313	4.3275	46.1197	4.4951
28	32.1574	4.0714	35.3383	4.3403	38.5038	4.5871	41.6236	4.8097
29	28.0860	4.2343	30.9980	4.5572	33.9167	4.8624	36.8139	5.1464
30	23.8517	4.4037	26.4418	4.7851	29.0543	5.1541	31.6675	5.5067
31	19.4480	4.5798	21.6557	5.0244	23.9002	5.4634	26.1608	5.8922
32	14.8682	4.7630	16.6313	5.2756	18.4368	5.7911	20.2686	6.3046
33	10.1052	4.9535	11.3557	5.5394	12.6457	6.1387	13.9640	6.7459
34	5.1517	5.1517	5.8163	5.8163	6.5070	6.5070	7.2181	7.2181
35	0.0000		0.0000		0.0000		0.0000	
		100.0000		100.0000		100.0000		100.0000

40-Year Life.

Age, in years.	Interest Rate. 4%		Interest Rate. 5%		Interest Rate. 6%		Interest Rate. 7%	
	Value.	Dep.	Value.	Dep.	Value.	Dep.	Value.	Dep.
0	100.0000		100.0000		100.0000		100.0000	
		1.0524		0.8278		0.6462		0.5009
1	98.9476		99.1722		99.3538		99.4991	
		1.0944		0.8692		0.6849		0.5360
2	97.8532		98.3030		98.6689		98.9631	
		1.1382		0.9127		0.7260		0.5735
3	96.7150		97.3903		97.9429		98.3896	
		1.1838		0.9583		0.7696		0.6136
4	95.5312		96.4320		97.1733		97.7760	
		1.2311		1.0062		0.8157		0.6566
5	94.3001		95.4258		96.3576		97.1194	
		1.2803		1.0565		0.8647		0.7026
6	93.0198		94.3693		95.4929		96.4168	
		1.3316		1.1093		0.9166		0.7517
7	91.6882		93.2600		94.5763		95.6651	
		1.3848		1.1648		0.9716		0.8044
8	90.3034		92.0952		93.6047		94.8607	
		1.4402		1.2230		1.0299		0.8606
9	88.8632		90.8722		92.5748		94.0001	
		1.4978		1.2842		1.0916		0.9210
10	87.3654		89.5880		91.4832		93.0791	
		1.5577		1.3484		1.1572		0.9853
11	85.8077		88.2396		90.3260		92.0938	
		1.6201		1.4158		1.2266		1.0544
12	84.1876		86.8238		89.0994		91.0394	
		1.6848		1.4866		1.3002		1.1281
13	82.5028		85.3372		87.7992		89.9113	
		1.7523		1.5610		1.3782		1.2071
14	80.7505		83.7762		86.4210		88.7042	
		1.8223		1.6390		1.4609		1.2917
15	78.9282		82.1372		84.9601		87.4125	
		1.8952		1.7210		1.5485		1.3820
16	77.0330		80.4162		83.4116		86.0304	
		1.9710		1.8070		1.6415		1.4788
17	75.0620		78.6092		81.7701		84.5517	
		2.0499		1.8974		1.7399		1.5823
18	73.0121		76.7118		80.0302		82.9694	
		2.1319		1.9923		1.8443		1.6930
19	70.8802		74.7195		78.1859		81.2764	
		2.2171		2.0919		1.9550		1.8116
20	68.6631		72.6276		76.2309		79.4648	
		2.3059		2.1965		2.0724		1.9384
21	66.3572		70.4311		74.1585		77.5264	
		2.3980		2.3063		2.1966		2.0741
22	63.9592		68.1248		71.9619		75.4523	
		2.4940		2.4216		2.3284		2.2192
23	61.4652		65.7032		69.6335		73.2331	
		2.5937		2.5427		2.4682		2.3746
24	58.8715		63.1605		67.1653		70.8585	
		2.6975		2.6698		2.6162		2.5408
25	56.1740		60.4907		64.5491		68.3177	
		2.8054		2.8033		2.7732		2.7187
26	53.3686		57.6874		61.7759		65.5990	
		2.9176		2.9434		2.9396		2.9090
27	50.4510		54.7440		58.8363		62.6900	
		3.0343		3.0906		3.1160		3.1126
28	47.4167		51.6534		55.7203		59.5774	
		3.1557		3.2451		3.3029		3.3305
29	44.2610		48.4083		52.4174		56.2469	
		3.2819		3.4074		3.5011		3.5636
30	40.9791		45.0009		48.9163		52.6833	
		3.4132		3.5778		3.7112		3.8131
31	37.5659		41.4231		45.2051		48.8702	
		3.5498		3.7567		3.9339		4.0800
32	34.0161		37.6664		41.2712		44.7902	
		3.6916		3.9445		4.1699		4.3656

40-Year Life.—(*Continued.*)

Age, in years.	Interest Rate. 4%		Interest Rate. 5%		Interest Rate. 6%		Interest Rate. 7%	
	Value.	Dep.	Value.	Dep.	Value.	Dep.	Value.	Dep.
33	30.3245	3.8393	33.7219	4.1417	37.1013	4.4200	40.4246	4.6712
34	26.4852	3.9930	29.5802	4.3488	32.6813	4.6853	35.7534	4.9982
35	22.4922	4.1526	25.2314	4.5662	27.9960	4.9664	30.7552	5.3480
36	18.3396	4.3189	20.6652	4.7946	23.0296	5.2644	25.4072	5.7225
37	14.0207	4.4915	15.8706	5.0343	17.7652	5.5802	19.6847	6.1229
38	9.5292	4.6712	10.8363	5.2860	12.1850	5.9151	13.5618	6.5516
39	4.8580	4.8580	5.5503	5.5503	6.2699	6.2699	7.0102	7.0102
40	0.0000		0.0000		0.0000		0.0000	
		100.0000		100.0000		100.0000		100.0000

45-Year Life.

0	100.0000	0.8262	100.0000	0.6262	100.0000	0.4700	100.0000	0.3500
1	99.1738	0.8593	99.3738	0.6575	99.5300	0.4983	99.6500	0.3744
2	98.3145	0.8937	98.7163	0.6903	99.0317	0.5283	99.2756	0.4007
3	97.4208	0.9294	98.0260	0.7249	98.5034	0.5597	98.8749	0.4287
4	96.4914	0.9666	97.3011	0.7611	97.9437	0.5934	98.4462	0.4587
5	95.5248	1.0053	96.5400	0.7992	97.3503	0.6290	97.9875	0.4908
6	94.5195	1.0455	95.7408	0.8392	96.7213	0.6668	97.4967	0.5252
7	93.4740	1.0872	94.9016	0.8810	96.0545	0.7068	96.9715	0.5620
8	92.3868	1.1308	94.0206	0.9251	95.3477	0.7492	96.4095	0.6013
9	91.2560	1.1760	93.0955	0.9714	94.5985	0.7941	95.8082	0.6434
10	90.0800	1.2230	92.1241	1.0200	93.8044	0.8418	95.1648	0.6884
11	88.8570	1.2720	91.1041	1.0710	92.9626	0.8923	94.4764	0.7366
12	87.5850	1.3229	90.0331	1.1245	92.0703	0.9459	93.7398	0.7882
13	86.2621	1.3757	88.9086	1.1807	91.1244	1.0025	92.9516	0.8433
14	84.8864	1.4308	87.7279	1.2398	90.1219	1.0628	92.1083	0.9024
15	83.4556	1.4880	86.4881	1.3018	89.0591	1.1265	91.2059	0.9655
16	81.9676	1.5476	85.1863	1.3668	87.9326	1.1941	90.2404	1.0331
17	80.4200	1.6094	83.8195	1.4352	86.7385	1.2657	89.2073	1.1055
18	78.8106	1.6738	82.3843	1.5070	85.4728	1.3417	88.1018	1.1828
19	77.1368	1.7408	80.8773	1.5823	84.1311	1.4222	86.9190	1.2657
20	75.3960	1.8104	79.2950	1.6614	82.7089	1.5075	85.6533	1.3542
21	73.5856	1.8828	77.6336	1.7445	81.2014	1.5979	84.2991	1.4490

45-YEAR LIFE.—(*Continued.*)

Age, in years.	Interest Rate. 4%		Interest Rate. 5%		Interest Rate. 6%		Interest Rate. 7%	
	Value.	Dep.	Value.	Dep.	Value.	Dep.	Value.	Dep.
22	71.7028	1.9581	75.8891	1.8317	79.6085	1.6939	82.8501	1.5504
23	69.7447	2.0865	74.0574	1.9233	77.9096	1.7954	81.2997	1.6590
24	67.7082	2.1180	72.1341	2.0195	76.1142	1.9083	79.6407	1.7751
25	65.5902	2.2026	70.1146	2.1205	74.2109	2.0174	77.8656	1.8994
26	63.3876	2.2907	67.9941	2.2264	72.1935	2.1388	75.9662	2.0323
27	61.0969	2.3824	65.7677	2.3378	70.0552	2.2668	73.9339	2.1746
28	58.7145	2.4777	63.4299	2.4548	67.7884	2.4028	71.7593	2.3268
29	56.2368	2.5768	60.9751	2.5773	65.3856	2.5469	69.4325	2.4897
30	53.6600	2.6798	58.3978	2.7062	62.8387	2.6997	66.9428	2.6640
31	50.9802	2.7870	55.6916	2.8417	60.1390	2.8617	64.2788	2.8504
32	48.1932	2.8986	52.8499	2.9837	57.2773	3.0334	61.4284	3.0499
33	45.2946	3.0144	49.8662	3.1328	54.2439	3.2155	58.3785	3.2635
34	42.2802	3.1350	46.7334	3.2896	51.0284	3.4088	55.1150	3.4920
35	39.1452	3.2605	43.4438	3.4539	47.6201	3.6129	51.6230	3.7363
36	35.8847	3.3909	39.9899	3.6267	44.0072	3.8296	47.8867	3.9979
37	32.4938	3.5264	36.3632	3.8081	40.1776	4.0594	43.8888	4.2777
38	28.9674	3.6676	32.5551	3.9988	36.1182	4.3029	39.6111	4.5772
39	25.2998	3.8142	28.5568	4.1984	31.8153	4.5612	35.0339	4.8976
40	21.4856	3.9669	24.3584	4.4088	27.2541	4.8348	30.1363	5.2404
41	17.5187	4.1255	19.9501	4.6286	22.4193	5.1249	24.8959	5.6073
42	13.3932	4.2905	15.3215	4.8600	17.2944	5.4323	19.2886	5.9997
43	9.1027	4.4621	10.4615	5.1032	11.8621	5.7584	13.2889	6.4197
44	4.6406	4.6406	5.3583	5.3583	6.1037	6.1037	6.8692	6.8692
45	0.0000		0.0000		0.0000		0.0000	
		100.0000		100.0000		100.0000		100.0000

50-YEAR LIFE.

0	100.0000	0.6550	100.0000	0.4777	100.0000	0.3444	100.0000	0.2460
1	99.3450	0.6812	99.5223	0.5015	99.6556	0.3651	99.7540	0.2632
2	98.6638	0.7085	99.0208	0.5267	99.2905	0.3870	99.4908	0.2816
3	97.9553	0.7368	98.4941	0.5529	98.9035	0.4102	99.2092	0.3014
4	97.2185	0.7663	97.9412	0.5806	98.4933	0.4349	98.9078	0.3224
5	96.4522	0.7969	97.3606	0.6097	98.0584	0.4609	98.5854	0.3450

50-YEAR LIFE.—(*Continued.*)

Age, in years.	Interest Rate. 4%		Interest Rate. 5%		Interest Rate. 6%		Interest Rate. 7%	
	Value.	Dep.	Value.	Dep.	Value.	Dep.	Value.	Dep.
6	95.6553		96.7509		97.5975		98.2404	
		0.8288		0.6401		0.4886		0.3692
7	94.8265		96.1108		97.1089		97.8712	
		0.8620		0.6722		0.5179		0.3950
8	93.9645		95.4386		96.5910		97.4762	
		0.8964		0.7057		0.5489		0.4226
9	93.0681		94.7329		96.0421		97.0536	
		0.9323		0.7410		0.5819		0.4522
10	92.1358		93.9919		95.4602		96.6014	
		0.9696		0.7781		0.6169		0.4839
11	91.1662		93.2138		94.8433		96.1175	
		1.0084		0.8170		0.6538		0.5178
12	90.1578		92.3968		94.1895		95.5997	
		1.0487		0.8578		0.6931		0.5540
13	89.1091		91.5390		93.4964		95.0457	
		1.0906		0.9008		0.7346		0.5928
14	88.0185		90.6382		92.7618		94.4529	
		1.1343		0.9457		0.7787		0.6343
15	86.8842		89.6925		91.9831		93.8186	
		1.1797		0.9931		0.8255		0.6786
16	85.7045		88.6994		91.1576		93.1400	
		1.2268		1.0427		0.8749		0.7262
17	84.4777		87.6567		90.2827		92.4138	
		1.2759		1.0948		0.9275		0.7771
18	83.2018		86.5619		89.3552		91.6367	
		1.3270		1.1496		0.9881		0.8314
19	81.8748		85.4123		88.3721		90.8053	
		1.3800		1.2071		1.0421		0.8896
20	80.4948		84.2052		87.3300		89.9157	
		1.4353		1.2674		1.1046		0.9519
21	79.0595		82.9378		86.2254		88.9638	
		1.4926		1.3307		1.1710		1.0185
22	77.5669		81.6017		85.0544		87.9453	
		1.5523		1.3974		1.2411		1.0898
23	76.0146		80.2097		83.8133		86.8555	
		1.6144		1.4672		1.3156		1.1661
24	74.4002		78.7425		82.4977		85.6894	
		1.6791		1.5405		1.3946		1.2477
25	72.7211		77.2020		81.1031		84.4417	
		1.7461		1.6176		1.4782		1.3351
26	70.9750		75.5844		79.6249		83.1066	
		1.8161		1.6984		1.5670		1.4285
27	69.1589		73.8860		78.0579		81.6781	
		1.8887		1.7884		1.6610		1.5285
28	67.2702		72.1026		76.3969		80.1496	
		1.9642		1.8726		1.7606		1.6355
29	65.3060		70.2300		74.6363		78.5141	
		2.0427		1.9661		1.8662		1.7500
30	63.2633		68.2639		72.7701		76.7641	
		2.1245		2.0645		1.9783		1.8725
31	61.1388		66.1994		70.7918		74.8916	
		2.2095		2.1677		2.0969		2.0086
32	58.9293		64.0317		68.6949		72.8880	
		2.2979		2.2761		2.2227		2.1438
33	56.6314		61.7556		66.4722		70.7442	
		2.3897		2.3899		2.3561		2.2939
34	54.2417		59.3657		64.1161		68.4503	
		2.4854		2.5095		2.4975		2.4545
35	51.7563		56.8562		61.6186		65.9958	
		2.5847		2.6347		2.6473		2.6263
36	49.1716		54.2215		58.9713		63.3695	
		2.6882		2.7666		2.8061		2.8101
37	46.4834		51.4549		56.1652		60.5594	
		2.7957		2.9050		2.9745		3.0068
38	43.6877		48.5499		53.1907		57.5526	
		2.9075		3.0501		3.1530		3.2173

50-YEAR LIFE.—(*Continued.*)

Age, in years.	Interest Rate. 4%		Interest Rate. 5%		Interest Rate. 6%		Interest Rate. 7%	
	Value.	Dep.	Value.	Dep.	Value.	Dep.	Value.	Dep.
39	40.7802		45.4998		50.0877		54.3353	
		3.0238		3.2027		3.3422		3.4425
40	37.7564		42.2971		46.6955		50.8928	
		3.1448		3.3629		3.5427		3.6835
41	34.6116		38.9342		43.1528		47.2093	
		3.2706		3.5309		3.7553		3.9414
42	31.3410		35.4033		39.3975		43.2679	
		3.4018		3.7075		3.9805		4.2172
43	27.9392		31.6958		35.4170		39.0507	
		3.5375		3.8929		4.2194		4.5124
44	24.4022		27.8029		31.1976		34.5388	
		3.6789		4.0875		4.4726		4.8284
45	20.7233		23.7154		26.7250		29.7099	
		3.8261		4.2919		4.7410		5.1662
46	16.8972		19.4235		21.9840		24.5437	
		3.9791		4.5065		5.0253		5.5280
47	12.9181		14.9170		16.9587		19.0157	
		4.1383		4.7319		5.3270		5.9149
48	8.7798		10.1851		11.6317		13.1008	
		4.3038		4.9683		5.6465		6.3289
49	4.4760		5.2168		5.9852		6.7719	
		4.4760		5.2168		5.9852		6.7719
50	0.0000		0.0000		0.0000		0.0000	
		100.0000		100.0000		100.0000		100.0000

60-YEAR LIFE.

	100.0000		100.0000		100.0000		100.0000	
		0.4202		0.2828		0.1876		0.1229
1	99.5798		99.9172		99.8124		99.8771	
		0.4370		0.2970		0.1988		0.1316
2	99.1428		99.4202		99.6136		99.7455	
		0.4544		0.3118		0.2108		0.1407
3	98.6884		99.1084		99.4028		99.6048	
		0.4727		0.3274		0.2234		0.1506
4	98.2157		98.7810		99.1794		99.4542	
		0.4916		0.3437		0.2368		0.1611
5	97.7241		98.4373		98.9426		99.2931	
		0.5112		0.3610		0.2510		0.1724
6	97.2129		98.0763		98.6916		99.1207	
		0.5317		0.3790		0.2661		0.1845
7	96.6812		97.6973		98.4255		98.9362	
		0.5529		0.3980		0.2820		0.1974
8	96.1283		97.2993		98.1435		98.7388	
		0.5750		0.4178		0.2989		0.2112
9	95.5533		96.8815		97.8446		98.5276	
		0.5981		0.4388		0.3169		0.2260
10	94.9552		96.4427		97.5277		98.3016	
		0.6220		0.4606		0.3360		0.2418
11	94.3332		95.9821		97.1917		98.0598	
		0.6468		0.4837		0.3560		0.2587
12	93.6864		95.4984		96.8357		97.8011	
		0.6728		0.5079		0.3775		0.2768
13	93.0136		94.9905		96.4582		97.5243	
		0.6996		0.5333		0.4000		0.2963
14	92.3140		94.4572		96.0582		97.2280	
		0.7276		0.5600		0.4241		0.3169
15	91.5864		93.8972		95.6341		96.9111	
		0.7567		0.5880		0.4495		0.3392
16	90.8297		93.3092		95.1846		96.5719	
		0.7870		0.6174		0.4765		0.3629
17	90.0427		92.6918		94.7081		96.2090	
		0.8185		0.6481		0.5051		0.3883

60-YEAR LIFE.—(*Continued*).

Age, in years.	Interest Rate. 4%		Interest Rate. 5%		Interest Rate. 6%		Interest Rate. 7%	
	Value.	Dep.	Value.	Dep.	Value.	Dep.	Value.	Dep.
18	89.2242	0.8512	92.0437	0.6807	94.2030	0.5354	95.8207	0.4154
19	88.3730	0.8853	91.3630	0.7146	93.6671	0.5675	95.4053	0.4446
20	87.4877	0.9207	90.6484	0.7504	93.1001	0.6016	94.9607	0.4757
21	86.5670	0.9575	89.8980	0.7880	92.4985	0.6377	94.4850	0.5089
22	85.6095	0.9958	89.1100	0.8273	91.8608	0.6759	93.9761	0.5446
23	84.6137	1.0356	88.2827	0.8686	91.1849	0.7165	93.4315	0.5828
24	83.5781	1.0771	87.4141	0.9122	90.4684	0.7594	92.8487	0.6235
25	82.5010	1.1201	86.5019	0.9577	89.7090	0.8051	92.2252	0.6671
26	81.3809	1.1650	85.5442	1.0056	88.9039	0.8533	91.5581	0.7139
27	80.2159	1.2115	84.5386	1.0559	88.0506	0.9045	90.8442	0.7638
28	79.0044	1.2600	83.4827	1.1087	87.1461	0.9589	90.0804	0.8178
29	77.7444	1.3104	82.3740	1.1641	86.1872	1.0168	89.2631	0.8745
30	76.4340	1.3629	81.2099	1.2223	85.1709	1.0778	88.3886	0.9357
31	75.0711	1.4173	79.9876	1.2835	84.0936	1.1420	87.4529	1.0012
32	73.6538	1.4741	78.7041	1.3476	82.9516	1.2104	86.4517	1.0714
33	72.1797	1.5329	77.3565	1.4149	81.7412	1.2881	85.3803	1.1462
34	70.6468	1.5944	75.9416	1.4858	80.4581	1.3601	84.2341	1.2266
35	69.0524	1.6580	74.4558	1.5600	79.0980	1.4417	83.0075	1.3124
36	67.3944	1.7245	72.8958	1.6380	77.6563	1.5282	81.6951	1.4043
37	65.6699	1.7934	71.2578	1.7200	76.1281	1.6199	80.2908	1.5025
38	63.8765	1.8651	69.5378	1.8059	74.5082	1.7171	78.7883	1.6078
39	62.0114	1.9397	67.7319	1.8962	72.7911	1.8201	77.1805	1.7203
40	60.0717	2.0173	65.8357	1.9911	70.9710	1.9293	75.4602	1.8407
41	58.0544	2.0980	63.8446	2.0906	69.0417	2.0452	73.6195	1.9695
42	55.9564	2.1820	61.7540	2.1951	66.9965	2.1677	71.6500	2.1074
43	53.7744	2.2692	59.5589	2.3049	64.8288	2.2978	69.5426	2.2550
44	51.5052	2.3599	57.2540	2.4201	62.5310	2.4357	67.2876	2.4128
45	49.1453	2.4544	54.8339	2.5411	60.0953	2.5819	64.8748	2.5816
46	46.6909	2.5526	52.2928	2.6682	57.5134	2.7387	62.2932	2.7625
47	44.1383	2.6546	49.6246	2.8016	54.7747	2.9010	59.5307	2.9557
48	41.4837	2.7609	46.8230	2.9416	51.8737	3.0750	56.5750	3.1627
49	38.7228	2.8712	43.8814	3.0888	48.8007	3.2596	53.4123	3.3841
50	35.8516	2.9861	40.7926	3.2432	45.5411	3.4551	50.0282	3.6209

60-YEAR LIFE.—(*Continued.*)

Age, in years.	Interest Rate. 4%		Interest Rate. 5%		Interest Rate. 6%		Interest Rate. 7%	
	Value.	Dep.	Value.	Dep.	Value.	Dep.	Value.	Dep.
51	32.8655	3.1056	37.5494	3.4053	42.0860	3.6624	46.4073	3.8745
52	29.7599	3.2298	34.1441	3.5756	38.4236	3.8821	42.5328	4.1456
53	26.5301	3.3590	30.5685	3.7544	34.5415	4.1151	38.3872	4.4358
54	23.1711	3.4933	26.8141	3.9421	30.4264	4.3620	33.9514	4.7463
55	19.6778	3.6331	22.8720	4.1392	26.0644	4.6237	29.2051	5.0785
56	16.0447	3.7784	18.7328	4.3462	21.4407	4.9012	24.1266	5.4341
57	12.2663	3.9295	14.3866	4.5635	16.5395	5.1952	18.6925	5.8145
58	8.3368	4.0867	9.8231	4.7917	11.3443	5.5069	12.8780	6.2214
59	4.2501	4.2501	5.0314	5.0314	5.8374	5.8374	6.6566	6.6566
60	0.0000		0.0000		0.0000		0.0000	
		100.0000		100.0000		100.0000		100.0000

80-YEAR LIFE.

0	100.0000	0.1614	100.0000	0.1030	100.0000	0.0573	100.0000	0.0314
1	99.8186	0.1887	99.8970	0.1081	99.9427	0.0607	99.9686	0.0336
2	99.6299	0.1962	99.7889	0.1135	99.8820	0.0643	99.9350	0.0359
3	99.4337	0.2041	99.6754	0.1192	99.8177	0.0682	99.8991	0.0384
4	99.2296	0.2122	99.5562	0.1252	99.7495	0.0723	99.8607	0.0411
5	99.0174	0.2207	99.4310	0.1314	99.6772	0.0766	99.8196	0.0440
6	98.7967	0.2295	99.2996	0.1380	99.6006	0.0812	99.7756	0.0471
7	98.5672	0.2387	99.1616	0.1449	99.5194	0.0861	99.7285	0.0504
8	98.3285	0.2483	99.0167	0.1521	99.4333	0.0913	99.6781	0.0539
9	98.0802	0.2582	98.8646	0.1597	99.3420	0.0967	99.6242	0.0576
10	97.8220	0.2685	98.7049	0.1677	99.2453	0.1025	99.5666	0.0617
11	97.5535	0.2793	98.5372	0.1761	99.1428	0.1087	99.5049	0.0660
12	97.2742	0.2904	98.3611	0.1849	99.0341	0.1152	99.4389	0.0706
13	96.9838	0.3021	98.1762	0.1941	98.9189	0.1221	99.3683	0.0756
14	96.6817	0.3141	97.9821	0.2039	98.7968	0.1294	99.2927	0.0809
15	96.3676	0.3267	97.7782	0.2140	98.6674	0.1372	99.2118	0.0865
16	96.0409	0.3398	97.5642	0.2248	98.5302	0.1454	99.1253	0.0926
17	95.7011	0.3534	97.3394	0.2360	98.3848	0.1542	99.0327	0.0990
18	95.3477	0.3675	97.1034	0.2478	98.2306	0.1634	98.9337	0.1060

80-YEAR LIFE.—(*Continued.*)

Age, in years.	Interest Rate. 4%		Interest Rate. 5%		Interest Rate. 6%		Interest Rate. 7%	
	Value.	Dep.	Value.	Dep.	Value.	Dep.	Value.	Dep.
19	94.9802		96.8556		98.0672		98.8277	
		0.3822		0.2602		0.1732		0.1184
20	94.5980		96.5954		97.8940		98.7143	
		0.3975		0.2732		0.1836		0.1213
21	94.2005		96.3222		97.7104		98.5930	
		0.4134		0.2869		0.1946		0.1298
22	93.7871		96.0353		97.5158		98.4632	
		0.4299		0.3012		0.2063		0.1389
23	93.3572		95.7341		97.3095		98.3243	
		0.4471		0.3162		0.2187		0.1485
24	92.9101		95.4179		97.0908		98.1758	
		0.4650		0.3321		0.2318		0.1590
25	92.4451		95.0858		96.8590		98.0168	
		0.4836		0.3487		0.2457		0.1702
26	91.9615		94.7371		96.6133		97.8466	
		0.5030		0.3661		0.2605		0.1821
27	91.4585		94.3710		96.3528		97.6645	
		0.5231		0.3844		0.2761		0.1948
28	90.9354		93.9866		96.0767		97.4697	
		0.5440		0.4036		0.2927		0.2085
29	90.3914		93.5830		95.7840		97.2612	
		0.5658		0.4238		0.3102		0.2231
30	89.8256		93.1592		95.4738		97.0381	
		0.5884		0.4450		0.3288		0.2387
31	89.2372		92.7142		95.1450		96.7994	
		0.6119		0.4672		0.3486		0.2554
32	88.6253		92.2470		94.7964		96.5440	
		0.6364		0.4906		0.3695		0.2733
33	87.9889		91.7564		94.4269		96.2707	
		0.6618		0.5151		0.3917		0.2924
34	87.3271		91.2413		94.0352		95.9783	
		0.6883		0.5409		0.4152		0.3129
35	86.6388		90.7004		93.6200		95.6654	
		0.7159		0.5679		0.4401		0.3348
36	85.9229		90.1325		93.1799		95.3306	
		0.7445		0.5963		0.4665		0.3582
37	85.1784		89.5362		92.7134		94.9724	
		0.7743		0.6262		0.4945		0.3833
38	84.4041		88.9100		92.2189		94.5891	
		0.8052		0.6575		0.5241		0.4101
39	83.5989		88.2525		91.6948		94.1790	
		0.8375		0.6903		0.5556		0.4388
40	82.7614		87.5622		91.1392		93.7402	
		0.8709		0.7249		0.5889		0.4696
41	81.8905		86.8373		90.5503		93.2706	
		0.9058		0.7611		0.6242		0.5024
42	80.9847		86.0762		89.9261		92.7682	
		0.9420		0.7992		0.6617		0.5376
43	80.0427		85.2770		89.2644		92.2306	
		0.9797		0.8391		0.7014		0.5752
44	79.0630		84.4379		88.5630		91.6554	
		1.0189		0.8811		0.7435		0.6155
45	78.0441		83.5568		87.8195		91.0399	
		1.0596		0.9251		0.7881		0.6586
46	76.9845		82.6317		87.0314		90.3813	
		1.1020		0.9714		0.8354		0.7047
47	75.8825		81.6603		86.1960		89.6766	
		1.1461		1.0199		0.8855		0.7540
48	74.7364		80.6404		85.3105		88.9226	
		1.1919		1.0709		0.9386		0.8068
49	73.5445		79.5695		84.3719		88.1158	
		1.2396		1.1245		0.9949		0.8633
50	72.3049		78.4450		83.3770		87.2525	
		1.2892		1.1807		1.0546		0.9237
51	71.0157		77.2643		82.3224		86.3288	
		1.3408		1.2397		1.1179		0.9883

80-YEAR LIFE.—(*Continued.*)

Age, in years.	Interest Rate. 4%		Interest Rate. 5%		Interest Rate. 6%		Interest Rate. 7%	
	Value.	Dep.	Value.	Dep.	Value.	Dep.	Value.	Dep.
52	69.6749	1.3944	76.0246	1.3017	81.2045	1.1850	85.3405	1.0575
53	68.2805	1.4502	74.7229	1.3668	80.0195	1.2561	84.2830	1.1316
54	66.8303	1.5082	73.3561	1.4352	78.7634	1.3314	83.1514	1.2108
55	65.3221	1.5685	71.9209	1.5069	77.4320	1.4113	81.9406	1.2955
56	63.7536	1.6312	70.4140	1.5823	76.0207	1.4960	80.6451	1.3862
57	62.1224	1.6965	68.8317	1.6614	74.5247	1.5858	79.2589	1.4832
58	60.4259	1.7644	67.1703	1.7445	72.9389	1.6809	77.7757	1.5871
59	58.6615	1.8349	65.4258	1.8317	71.2580	1.7818	76.1886	1.6982
60	56.8266	1.9083	63.5941	1.9233	69.4762	1.8887	74.4904	1.8170
61	54.9183	1.9847	61.6708	2.0194	67.5875	2.0020	72.6734	1.9442
62	52.9336	2.0640	59.6514	2.1204	65.5855	2.1221	70.7292	2.0803
63	50.8696	2.1466	57.5310	2.2264	63.4634	2.2495	68.6489	2.2259
64	48.7230	2.2325	55.3046	2.3377	61.2139	2.3844	66.4230	2.3818
65	46.4905	2.3218	52.9669	2.4546	58.8295	2.5275	64.0412	2.5485
66	44.1687	2.4147	50.5123	2.5773	56.3020	2.6791	61.4927	2.7269
67	41.7540	2.5112	47.9350	2.7062	53.6229	2.8399	58.7658	2.9178
68	39.2428	2.6117	45.2288	2.8415	50.7830	3.0108	55.8480	3.1220
69	36.6311	2.7162	42.3873	2.9836	47.7727	3.1909	52.7260	3.3405
70	33.9149	2.8248	39.4037	3.1328	44.5818	3.3823	49.3855	3.5744
71	31.0901	2.9378	36.2709	3.2894	41.1995	3.5853	45.8111	3.8246
72	28.1523	3.0553	32.9815	3.4539	37.6142	3.8004	41.9865	4.0923
73	25.0970	3.1775	29.5276	3.6266	33.8138	4.0284	37.8942	4.3788
74	21.9195	3.3046	25.9010	3.8079	29.7854	4.2701	33.5154	4.6853
75	18.6149	3.4368	22.0931	3.9983	25.5153	4.5263	28.8301	5.0133
76	15.1781	3.5743	18.0948	4.1982	20.9890	4.7979	23.8168	5.3642
77	11.6038	3.7173	13.8966	4.4081	16.1911	5.0858	18.4526	5.7397
78	7.8865	3.8659	9.4885	4.6285	11.1053	5.3909	12.7129	6.1415
79	4.0206	4.0206	4.8600	4.8600	5.7144	5.7144	6.5714	6.5714
80	0.0000		0.0000		0.0000		0.0000	
		100.0000		100.0000		100.0000		100.0000

100-YEAR LIFE.

0	100.0000	0.0808	100.0000	0.0383	100.0000	0.0177	100.0000	0.0081
1	99.9192	0.0840	99.9617	0.0402	99.9823	0.0188	99.9919	0.0086
2	99.8352	0.0874	99.9215	0.0423	99.9635	0.0199	99.9833	0.0092

100-YEAR LIFE.—(*Continued.*)

Age, in years.	Interest Rate. 4%		Interest Rate. 5%		Interest Rate. 6%		Interest Rate. 7%	
	Value.	Dep.	Value.	Dep.	Value.	Dep.	Value.	Dep.
3	99.7478	0.0909	99.8792	0.0444	99.9436	0.0211	99.9741	0.0099
4	99.6569	0.0945	99.8348	0.0466	99.9225	0.0224	99.9642	0.0106
5	99.5624	0.0983	99.7882	0.0489	99.9001	0.0237	99.9536	0.0113
6	99.4641	0.1022	99.7393	0.0514	99.8764	0.0251	99.9423	0.0121
7	99.3619	0.1063	99.6879	0.0539	99.8513	0.0267	99.9302	0.0130
8	99.2556	0.1106	99.6340	0.0566	99.8246	0.0283	99.9172	0.0139
9	99.1450	0.1150	99.5774	0.0594	99.7963	0.0300	99.9033	0.0148
10	99.0300	0.1196	99.5180	0.0624	99.7663	0.0318	99.8885	0.0159
11	98.9104	0.1244	99.4556	0.0655	99.7345	0.0337	99.8726	0.0170
12	98.7860	0.1294	99.3901	0.0688	99.7008	0.0357	99.8556	0.0182
13	98.6566	0.1345	99.3213	0.0723	99.6651	0.0378	99.8374	0.0195
14	98.5221	0.1399	99.2490	0.0759	99.6273	0.0401	99.8179	0.0208
15	98.3822	0.1455	99.1731	0.0797	99.5872	0.0425	99.7971	0.0223
16	98.2367	0.1513	99.0934	0.0836	99.5447	0.0450	99.7748	0.0238
17	98.0854	0.1574	99.0098	0.0878	99.4997	0.0477	99.7510	0.0255
18	97.9280	0.1637	98.9220	0.0922	99.4520	0.0506	99.7255	0.0273
19	97.7643	0.1702	98.8298	0.0968	99.4014	0.0537	99.6982	0.0292
20	97.5941	0.1770	98.7330	0.1017	99.3477	0.0569	99.6690	0.0313
21	97.4171	0.1841	98.6313	0.1067	99.2908	0.0603	99.6377	0.0334
22	97.2330	0.1915	98.5246	0.1121	99.2305	0.0639	99.6043	0.0358
23	97.0415	0.1992	98.4125	0.1177	99.1666	0.0677	99.5685	0.0383
24	96.8423	0.2071	98.2948	0.1236	99.0989	0.0718	99.5302	0.0410
25	96.6352	0.2154	98.1712	0.1298	99.0271	0.0761	99.4892	0.0438
26	96.4198	0.2240	98.0414	0.1362	98.9510	0.0807	99.4454	0.0469
27	96.1958	0.2330	97.9052	0.1431	98.8703	0.0855	99.3985	0.0502
28	95.9628	0.2423	97.7621	0.1502	98.7848	0.0906	99.3483	0.0537
29	95.7205	0.2520	97.6119	0.1577	98.6942	0.0961	99.2946	0.0575
30	95.4685	0.2621	97.4542	0.1656	98.5981	0.1019	99.2371	0.0615
31	95.2064	0.2726	97.2886	0.1739	98.4962	0.1080	99.1756	0.0658
32	94.9338	0.2835	97.1147	0.1826	98.3882	0.1145	99.1098	0.0704
33	94.6503	0.2948	96.9321	0.1917	98.2737	0.1213	99.0394	0.0753
34	94.3555	0.3066	96.7404	0.2013	98.1524	0.1286	98.9641	0.0806
35	94.0489	0.3188	96.5391	0.2113	98.0238	0.1363	98.8835	0.0862
36	93.7301	0.3316	96.3278	0.2219	97.8875	0.1445	98.7973	0.0923

100-YEAR LIFE.—(*Continued.*)

Age, in years.	Interest Rate. 4%		Interest Rate. 5%		Interest Rate. 6%		Interest Rate. 7%	
	Value.	Dep.	Value.	Dep.	Value.	Dep.	Value.	Dep.
37	93.3985	0.3449	96.1059	0.2330	97.7480	0.1532	98.7050	0.0987
38	93.0536	0.3587	95.8729	0.2447	97.5898	0.1624	98.6063	0.1056
39	92.6949	0.3730	95.6282	0.2569	97.4274	0.1721	98.5007	0.1130
40	92.3219	0.3879	95.3718	0.2697	97.2553	0.1824	98.3877	0.1209
41	91.9340	0.4034	95.1016	0.2832	97.0729	0.1934	98.2668	0.1294
42	91.5306	0.4196	94.8184	0.2974	96.8795	0.2050	98.1374	0.1385
43	91.1110	0.4364	94.5210	0.3123	96.6745	0.2173	97.9989	0.1482
44	90.6746	0.4538	94.2087	0.3279	96.4572	0.2303	97.8507	0.1585
45	90.2208	0.4720	93.8808	0.3443	96.2269	0.2441	97.6922	0.1696
46	89.7488	0.4908	93.5365	0.3615	95.9828	0.2588	97.5226	0.1815
47	89.2580	0.5105	93.1750	0.3795	95.7240	0.2743	97.3411	0.1942
48	88.7475	0.5309	92.7955	0.3985	95.4497	0.2908	97.1469	0.2078
49	88.2166	0.5521	92.3970	0.4184	95.1589	0.3082	96.9391	0.2223
50	87.6645	0.5742	91.9786	0.4394	94.8507	0.3267	96.7168	0.2379
51	87.0903	0.5972	91.5392	0.4613	94.5240	0.3463	96.4789	0.2546
52	86.4931	0.6211	91.0779	0.4844	94.1777	0.3671	96.2243	0.2724
53	85.8720	0.6459	90.5935	0.5086	93.8106	0.3891	95.9519	0.2915
54	85.2261	0.6718	90.0849	0.5340	93.4215	0.4124	95.6604	0.3119
55	84.5543	0.6986	89.5509	0.5607	93.0091	0.4372	95.3485	0.3337
56	83.8557	0.7266	88.9902	0.5888	92.5719	0.4634	95.0148	0.3570
57	83.1291	0.7556	88.4014	0.6182	92.1085	0.4912	94.6578	0.3820
58	82.3735	0.7859	87.7832	0.6491	91.6173	0.5207	94.2758	0.4088
59	81.5876	0.8173	87.1341	0.6816	91.0966	0.5519	93.8670	0.4374
60	80.7703	0.8500	86.4525	0.7157	90.5447	0.5851	93.4296	0.4680
61	79.9203	0.8840	85.7368	0.7515	89.9596	0.6202	92.9616	0.5008
62	79.0363	0.9193	84.9853	0.7890	89.3394	0.6574	92.4608	0.5358
63	78.1170	0.9561	84.1963	0.8285	88.6820	0.6968	91.9250	0.5733
64	77.1609	0.9944	83.3678	0.8699	87.9852	0.7386	91.3517	0.6135
65	76.1665	1.0341	82.4979	0.9134	87.2466	0.7829	90.7382	0.6564
66	75.1324	1.0755	81.5845	0.9591	86.4637	0.8299	90.0818	0.7024
67	74.0579	1.1185	80.6254	1.0070	85.6338	0.8797	89.3794	0.7515
68	72.9394	1.1633	79.6184	1.0574	84.7541	0.9325	88.6279	0.8041
69	71.7751	1.2098	78.5610	1.1102	83.8216	0.9884	87.8238	0.8604
70	70.5653	1.2582	77.4508	1.1657	82.8332	1.0477	86.9634	0.9206

100-Year Life.—(*Continued.*)

Age, in years.	Interest Rate. 4%		Interest Rate. 5%		Interest Rate. 6%		Interest Rate. 7%	
	Value.	Dep.	Value.	Dep.	Value.	Dep.	Value.	Dep.
71	69.3071	1.3085	75.2851	1.2240	81.7855	1.1106	86.0428	0.9851
72	67.9986	1.3608	75.0611	1.2852	80.6749	1.1772	85.0577	1.0540
73	66.6378	1.4153	73.7759	1.3495	79.4977	1.2479	84.0037	1.1278
74	65.2225	1.4719	72.4264	1.4170	78.2498	1.3228	82.8759	1.2068
75	63.7506	1.5308	71.0094	1.4878	76.9270	1.4022	81.6691	1.2912
76	62.2198	1.5920	69.5216	1.5622	75.5248	1.4863	80.3779	1.3816
77	60.6278	1.6557	67.9594	1.6408	74.0385	1.5754	78.9963	1.4783
78	58.9721	1.7219	66.3191	1.7228	72.4631	1.6700	77.5180	1.5818
79	57.2502	1.7908	64.5968	1.8084	70.7931	1.7701	75.9362	1.6925
80	55.4594	1.8624	62.7884	1.8989	69.0230	1.8764	74.2437	1.8110
81	53.5970	1.9369	60.8895	1.9938	67.1466	1.9889	72.4327	1.9378
82	51.6601	2.0144	58.8957	2.0935	65.1577	2.1083	70.4949	2.0734
83	49.6457	2.0950	56.8022	2.1982	63.0494	2.2348	68.4215	2.2186
84	47.5507	2.1788	54.6040	2.3081	60.8146	2.3689	66.2029	2.3739
85	45.3719	2.2659	52.2959	2.4235	58.4457	2.5110	63.8290	2.5400
86	43.1060	2.3566	49.8724	2.5447	55.9347	2.6616	60.2890	2.7179
87	40.7494	2.4508	47.3277	2.6719	53.2731	2.8213	58.5711	2.9081
88	38.2986	2.5489	44.6558	2.8055	50.4518	2.9906	55.6630	3.1117
89	35.7497	2.6508	41.8503	2.9458	47.4612	3.1701	52.5513	3.3295
90	33.0989	2.7568	38.9045	3.0931	44.2911	3.3603	49.2218	3.5625
91	30.3421	2.8671	35.8114	3.2477	40.9308	3.5619	45.6593	3.8119
92	27.4750	2.9818	32.5637	3.4101	37.3689	3.7756	41.8474	4.0788
93	24.4932	3.1011	29.1536	3.5806	33.5933	4.0021	37.7686	4.3643
94	21.3921	3.2251	25.5730	3.7597	29.5912	4.2423	33.4043	4.6698
95	18.1670	3.3541	21.8133	3.9477	25.3489	4.4968	28.7345	4.9967
96	14.8129	3.4883	17.8656	4.1450	20.8521	4.7666	23.7378	5.3464
97	11.3246	3.6278	13.7206	4.3523	16.0855	5.0526	18.3914	5.7207
98	7.6968	3.7729	9.3683	4.5699	11.0329	5.3558	12.6707	6.1211
99	3.9239	3.9239	4.7984	4.7984	5.6771	5.6771	6.5496	6.5496
100	0.0000		0.0000		0.0000		0.0000	
		100.0000		100.0000		100.0000		100.0000

APPENDIX II.

Some Examples of the Expectation of Life of So-Called Permanent Structures.

When one examines records of the life of different kinds of so-called permanent structures, the actual life of such structures is frequently found to be much shorter than the life usually estimated, and probably much shorter than was anticipated when the works were new. For the purpose of illustration, this subject has been investigated along two lines: (*a*) the actual life of railway stations, and especially of terminal stations; (*b*) the life of water-works sources, reservoirs, and pumping stations.

TABLE 5.—Life of Railway Stations.

Location.	Year when built or enlarged.	Life, in years.	Remarks.
Grand Central Station and Predecessors, New York City.			
241 Bowery, at Prince Street	1832	7	
Tryon Row (Municipal) Building	1839		
Fourth Avenue and Twenty-sixth Street (Madison Square Garden)	1857	18	
Forty-second Street	1873	16	
Forty-second Street. Extensive enlargement	1884		
Forty-second Street Station. Entirely remodeled and three stories added for offices	1899	34	
Construction of new terminal started	1903		
Present station first used for electric service	1907		
Present station in full use	1912		
Boston and Albany Railroad, Albany, N. Y.			
Wooden building at Colonie and Broadway	1866	7	
Brick and stone building at Maiden Lane	1873		
Existing stone station	1900	27	
Boston and Albany Railroad, Boston, Mass.			
Original station	1834		History between these dates not known.
Kneeland Street Station	1881	18	
South Station (Union Station)	1899		
Boston and Providence Railroad, Boston, Mass.			
Original station	1834		History not known.
Park Square Station	1874	25	
South Station	1899		

TABLE 5.—LIFE OF RAILWAY STATIONS.—(*Continued.*)

Location.	Year when built or enlarged.	Life, in years.	Remarks.
NEW YORK AND NEW ENGLAND RAILROAD, BOSTON, MASS.			
Original station	1855	17	
Station burned and rebuilt	1872		
Station extensively remodeled	1880	27	
South Station	1899		
BOSTON AND MAINE RAILROAD, BOSTON, MASS.			
Haymarket Square Station	1846	47	
North Station (Union Station)	1893		
EASTERN RAILROAD, BOSTON, MASS.			
Causeway Street Station	1854	9	
Station burned and replaced by brick building	1863	30	
North Station	1893		
FITCHBURG RAILROAD, BOSTON, MASS.			
Causeway Street Station	1848	45	
North Station	1893		
BOSTON AND LOWELL RAILROAD, BOSTON, MASS.			
Minot Street Station	1835	17	
Causeway Street Station	1852		
Causeway Street Station rebuilt	1873	21	
Merged into North Station, with minor alterations	1893		
BOSTON AND ALBANY RAILROAD, WORCESTER, MASS.			
Original station	1835	36	History not known.
Union Station	1875		
New Union Station	1911		
Average life of all stations		24	

The Forty-second Street Station, in New York City, although credited with a total life of 34 years, had that length of life only for the part built in 1883. The extensive enlargement in 1884 had a life of 23 years and the remodeled station of 1899, had a life of but 8 years.

Life of Water-Works Structures.

The average life of the sources of water supply given in Table 6 was shortened by the introduction of a general system of water supply for the Metropolitan District, which made the further use of small local supplies undesirable.

Lake Cochituate, the original source of supply of Boston, has been in use 65 years, and the Sudbury River, the second source of supply of the city, has been in use 36 years, but both these sources are now used only to a limited extent to supplement, when necessary, the supply from the Wachusett Reservoir, the newest source.

TABLE 6.—Sources of Supply.
Massachusetts Metropolitan Water District.

Source.	Period. Years.	Life, in years.	Remarks.
Boston, Mystic Lake source, supplying 12 000 000 gal. daily in the last year of its life	1864-1898	34	Abandoned on account of increasing pollution.
Malden, Spot Pond	1870-1898	28	
Malden, ground-water source	1890-1900	10	
Quincy, ground-water source	1884-1898	14	
Quincy, storage reservoir	1888-1898	10	
Hyde Park, ground-water source, near Neponset River	1885-1912	27	
Hyde Park, ground-water source, near Mother Brook	1899-1912	13	
Medford, Spot Pond	1870-1898	28	
Medford, storage reservoir	1894-1899	5	
Revere, ground-water source in town	1884-1898	14	
Revere, ground-water source in Cliftondale	1891-1899	8	
Melrose, Spot Pond	1870-1898	28	
Watertown, ground-water source	1885-1898	13	
Arlington, storage reservoir	1872-1899	27	
Arlington, ground-water source	1895-1900	5	
Swampscott, ground-water source	1885-1899	14	
Lexington, ground-water source	1884-1903	19	
Lexington, storage reservoir	1894-1903	9	
Average		17	

Reservoirs.

At the time of the completion of the original works supplying Boston, in 1848, there was a reservoir of 23 acres in Brookline at the end of the brick aqueduct. The pipes from this reservoir led to three distributing reservoirs: one of them, on Beacon Hill in Boston, was a large elevated masonry reservoir supported by arches; the other two were earthen reservoirs on hills in South Boston and East Boston.

The Brookline Reservoir lost a large part of its value at the end of 22 years, when it was superseded by a larger reservoir, but it had a life of 52 years before its use was discontinued. The use of Beacon Hill Reservoir was discontinued in 23 years; of the South Boston

Reservoir in 24 years; and of the East Boston Reservoir, except for the purpose of emergency use, in 32 years.

As in the case of the sources of supply, the life of pumping stations in the Massachusetts Metropolitan Water District, has, in many cases, been shortened by the introduction of a general system of supply. The oldest of the five existing pumping stations in this district was built 26 years ago, and the building was extended 11 years after it was built.

A study of other water systems shows that sources, pumping stations, reservoirs and other works are frequently abandoned for various reasons, such as the pollution of sources, the growth of population at higher elevations than those originally provided for, and the general improvement and enlargement of the water system to meet changing conditions.

An instance of an extremely short life of a reservoir and pumping station was observed in connection with a valuation for rate-making made in 1909. The rates in question were those of 1904 and 1905. The pumping station and reservoir did not appear on the inventory for 1904, because they had not then been built. They did appear on the inventory for 1905, but could not be seen by the appraiser in 1909 because they had been superseded and removed. The cause of the short life was a great influx of population which settled on elevated land and required a more complete system of high-service works.

TABLE 7.—Pumping Stations.

	Period. Years.	Life, in years.
Boston, original high service	1870–1888	18
Old East Boston	1880–1889	9
New East Boston	1889–1898	9
West Roxbury	1886–1913	27
Mystic { Worthington Pump	1864–1898	34
Mystic { Leavitt Pump*	1896–1898	2
Somerville	1890–1900	10
Malden, Spot Pond Station	1883–1898	15
Malden, Webster Park Station	1890–1900	10
Chelsea	1886–1900	14
Everett	1888–1899	11
Quincy	1884–1898	14
Hyde Park, Neponset River Station	1885–1912	27
Hyde Park, Mother Brook Station	1899–1912	13
Medford, Spot Pond Station	1892–1898	6
Medford, Reservoir Station	1894–1899	5
Revere, Town Station	1884–1898	14
Revere, Cliftondale Station	1891–1899	8
Melrose	1886–1899	13
Watertown	1885–1898	13
Arlington	1895–1900	5
Swampscott	1885–1899	14
Lexington	1881–1903	19
Average		14

*This pumping engine was transferred to another pumping station when the Mystic Station was abandoned.

On the other hand it is not to be inferred from Table 6, that the average life of sources of water supply is but 17 years, or from Table 7, that the average life of pumping stations is but 14 years—for the life history of the majority of water-works will show that such is not the case. These examples have been cited merely to indicate that functional depreciation is an active force to be considered and given such practical weight as circumstances and experience may indicate to be fair.

Thus, by comparison with Table 7, there might be cited the experience of the Spring Valley Water Company, of San Francisco, Cal. Out of eight pumping stations (excluding the centrifugal booster and ground-water supply stations, recently erected and still in service), all but one are in active service—and this one was an emergency station, and not one of permanent construction. The age of the oldest station is about 30 years, the mean age of the investment in the eight stations, 15 years.

Table 8 is a statement of the pumps in these stations.

TABLE 8.

Fly-wheel pumps.		Direct-acting pumps.	
Black Point	1-30 years.	Ocean View	2-18 years.
" "	2-22 "	Precita	1-17 "
Belmont	2-28 "	"	1- 8 "
"	3-15 "		
Merced	2-25 "	Centrifugal pumps.	
Millbrae	1-18 "	Ravenswood	1- 4 years.
Central	1-18 "	Pleasanton	1- 3 "
Clarendon	1-15 "	"	1- 7 "
		Crystal Springs	3- 3 "

No pumps have been discarded, except three small double-acting low-duty pumps at Bald Hill emergency station, used in two or three dry years for a few months only.

This comparison is made merely to indicate that each problem must be thoroughly studied in the light of the local past, present, and probable future conditions.

APPENDIX III.

Some Examples of Actual Overhead Cost.

In presenting these data as to actual overhead cost, the Committee desires to lay stress on the need of using all such figures with caution. Items entering into overhead cost of different and similar works are rarely alike, and many of the expenses, and even classes of expense, incurred on one piece of work, might not be incurred on another. It is impossible to forecast all the conditions and items of overhead cost which will develop on any work. Were it otherwise, no separate group of overhead cost would be necessary, for every expense would be included in the unit costs.

Experience has shown, however, that indeterminate or indeterminable expenses always occur, and in such amount that they frequently aggregate like, or similar, percentages of fixed unit costs. Hence, the mere amount of those percentages, developed on other works, has a bearing on the problem. Their segregation aids in forecasting probable developments the more accurately, and they furnish a background of comparative experience which is useful.

Moreover, in valuing properties, years after their construction, grave uncertainties generally arise as to the conditions which actually developed during the construction of these works, and records are virtually never in sufficient detail to eliminate them. Therefore, the evaluator, as the designing and constructing engineer, must make reasonable allowance, to cover the effect of these uncertainties, if justice is to be done.

To the evaluator, the records of greatest value are those developed on work, with all the details, conditions, and surroundings of which he is personally familiar, as these he is in a position to use most intelligently as a yard-stick, and is least likely to misapply. The evaluator should not attempt to figure the value of property with the construction costs of which he is not familiar.

With reference to the data submitted, the details presented to the Committee were so voluminous as to be impracticable of publication.

(*a*).—Examples Showing Cumulative Engineering Costs, with Varying Volumes of Work.

Compiled from printed reports, under the supervision of Alfred Craven, M. Am. Soc. C. E.

Year.	New York State Barge Canal.	Board of Water Supply—New York Aqueduct.
1905	750%	.....
1906	98	167.0%
1907	40	174.0
1908	25	81.3
1909	14	32.9
1910	13	19.2
1911	9	14.5
1912	9	12.5
1913	9	11.86

(*b*).—Examples of Overhead Costs on Water-Works Construction.

Administration, Engineering, Miscellaneous, Etc., Excluding Land (except as otherwise noted), Taxes, Interest-during-Construction, and Discount and Expenses on Bonds and other Securities Issued.

Compiled by Leonard Metcalf, M. Am. Soc. C. E.

Year to which expenditures have been estimated.	Locality and character of works.	Total construction cost excl. overhead and interest and land except as otherwise noted.	Administration.	Engineering and superintendence.	Police supervision.	Miscellaneous.	Total excluding taxes, contingencies, and interest.	Authority and source of information, and remarks.
1914	New York Catskill Supply, on structures only:							
	For engineering	$108 144 219	.....	11.12%	.....			*Excludes 0.5% Preliminary Engineering Report. Based on Annual Report of Board of Water Supply. Interest during construction, 18.02%, giving **total, 35 2%** (Hazen).
	" police	$99 393 000	.....	.....	2.18%	Excluded.	*14.56%	
	" administration	$113 861 751	1.26%	.....	.....			
	On land, including overhead and interest	$21 196 700	.....	.....	.....	.....	43.08%	Including interest and taxes to January 1st, 1916, when works go into service, 37.15%, the total overhead and interest = 96.23% (Hazen's analysis).
1912	Metropolitan Water-Works (Boston), excluding purchase of existing works	$18 058 000	.....	11.52%	1.17%	See note	.....	Leonard Metcalf and Charles W. Sherman, from 12th Annual Report, Met. W. & S. Bd.—no allowance for legal services by Attorney-General's office.
1912	Metropolitan Water-Works (Boston), Excluding purchase of existing works, **but including land damages, and water diversion**	$23 427 000	1.35%	9.55%	0.90%	1.08% (see note)	13.01 (Incomplete. See note).	Leonard Metcalf and Charles W. Sherman, from 12th Annual Report, Met. W. & S. Bd.—**no allowance** for legal service of Attorney-General's office.
1906	Metropolitan Water-Works:							
	Wachusett Dam	$2 014 000	.....	12.74%	.....	.....	.....	
	Wachusett Reservoir	4 030 000	.....	14.62	.....	.....	.....	
	Wachusett Aqueduct	1 537 000	.....	10.85*	.....	.....	.....	
			.....	9.33†	.....	.....	.....	
	Sudbury Reservoir	2 127 000	.....	7.08*	.....	.....	.....	
			.....	6.37†	.....	.....	.....	
	Weston Aqueduct	2 915 000	.....	10.65	.....	.....	.....	
	Improvement of Lake Cochituate	92 300	.....	10.4	.....	.....	.....	
	Main pipe lines	2 517 000	.....	8.63	.....	.....	.....	Leonard Metcalf and Charles W. Sherman, from ms. financial statement of Metropolitan Water Works to Jan. 1, 1907.
	Spot Pond Reservoir	487 500	.....	18.6	.....	.....	.....	
	Fells Reservoir	119 500	.....	18.3	.....	.....	.....	
	Bear Hill Reservoir	31 400	.....	22.0	.....	.....	.....	
	Forbes Hill Reservoir and Stand-pipe	73 400	.....	22.7	.....	.....	.....	
	Low-Service Pumping Station, Chestnut Hill	422 400	.....	9.5	.....	.....	.....	
	High-Service Pumping Station, Chestnut Hill	225 400	.....	7.5	.....	.....	.....	
	Spot Pond Pumping Station	269 200	.....	8.4	.....	.....	.....	
	Meters and Connections	74 300	.....	3.6	.....	.....	.....	
	Clinton Sewage Works	88 900	.....	25.5	.....	.....	.....	
1897-Dec. 31, 1907	Cincinnati Water-Works, Ohio	$17 024 300		11.6*				Hazen's analysis. Interest during construction 15.24%, making total overhead and interest **29.90%**.
		$9 469 918	.....	11.4†	.....	.....	12.72%	
	do. excluding real estate, overhead, and interest	$9 279 203	4.5%	9.1%	Legal	0.4%	14.0%	Metcalf incl. 0.9% prelim. engr. and experimental plant and 0.4% legal. Grand total cost excl. interest $10 844 331.
1910	Pittsfield, Mass., Water-Works	463 751	.....	13.8%	.....	.....	.....	Metcalf.
1906	Kennebec Water District, Waterville, Me.	267 408	2.53%	7.35%	.....	.....	.....	Metcalf.
1910	Springfield, Mass., Water-Works:							
	(*a*) New Little River Supply, dam, filters, conduits	2 000 000	.....	.....	.....	.....	14.30%	(Hazen) on new supply.
1906	(*b*) Ludlow filters	43 306	.....	17.0%	.....	.....	.....	Emergency supply.
1911	Des Moines, Ia., Water-Works, new source	.....	.....	8.6%	.....	.....	.....	Hazen.

* Including engineering connected with acquisition of real estate,

† After eliminating, by estimation, engineering connected with real estate.

(*c*).—Examples of Overhead Costs on Various Works.—(*Continued*).

Administration, Engineering, Miscellaneous, Etc., Excluding Taxes, Interest-during-Construction, and Discount and Expenses upon Bonds and other Securities Issued.

Compiled by Leonard Metcalf, M. Am. Soc. C. E.

Year.	Locality and character of works.	Total construction cost excl. overhead and int. and taxes.	Administration.	Engineering and superintendence.	General miscellaneous and incidental expense.	Total, excluding taxes, contingencies, and interest.	Authority, source of information, and remarks.
1867–1901.	St. Louis, Mo., Water-Works...	$8 472 576	.....	11.2%	.....	.....	Report on water supply of City of St. Louis, 1902, App. D., p. 139.
1905–1914.	Los Angeles, Cal., Aqueduct...	22 540 580	.....	8.8%	.....	.....	Note.—Of the total cost exclusive of interest ($24 526 868), 22.3% covered the auxiliary or incidental construction.
Slow Sand Filter Plants.							
1908.....	Watertown, N. Y., filters.....	$97 065	.....	5.9%	.....	.....	Allen Hazen.
1910.....	Ogdensburg, N. Y., filters.....	167 694	.....	7.0%	.....	.....	" "
1904.....	Hudson River State Hospital filters.....	36 000	.....	10.5%	.....	.....	" "
1908.....	Peekskill, N. Y., filters.....	63 306	.....	7.1%	.....	.....	" "
1903.....	Yonkers, N. Y., open filters.....	50 165	.....	7.3%	.....	.....	" "
1907.....	Yonkers, N. Y., covered filters..	106 708	.....	8.7%	.....	.....	" "
1903.....	Ithaca, N. Y.....	192 114	.....	7.0%	.....	.....	" "
1903–05...	Washington, D. C., filters.....	3 356 000	.....	7.4%	.....	.....	" "
1909–12...	Toronto, Canada, filters.....	781 519	.....	10.2%	.....	.....	(Excluding work done City Engineer's Corps.)
1897–99...	Albany, N. Y., filters.....	460 600	.....	6.7%	.....	.....	*Transactions*, Am. Soc. C. E., Vol. XLIII, p. 294.
1896.....	Ashland, Wis., filters.....	38 378	.....	4.7%	.....	.....	Excluding work of Company employees, and administration and miscellaneous. (William Wheeler.)
1904–08.	Columbus, Ohio, improved water-works.....	1 189 210	.....	8.1%	.....	.....	*Transactions*, Am. Soc. C. E., Vol. XLVII, p. 261.

(*c*).—Examples of Overhead Costs on Various Works.

Administration, Engineering, Miscellaneous, Etc., Excluding Taxes, Interest-during-Construction, and Discount and Expenses on Bonds and other Securities Issued.

Compiled by Leonard Metcalf, M. Am. Soc. C. E.

Year.	Locality and character of works.	Total construction cost, excl. overhead and int. and taxes.	Administration.	Engineering and superintendence.	General miscellaneous and incidental expense.	Total, excluding taxes, contingencies, and interest.	Authority, source of information, and remarks.
1909	Charles River Basin Commission, Boston. Dam, locks, embankments, conduits	$3 215 656	3.2%	15.5%	.....	18.7%	
1906-1912	Louisville, Ky. Sewers	$3 289 331	2.37%	1.70% Prel. 10.23%	1.09%	15.39%	Harrison P. Eddy.
1895-1912	Boston Transit Commission, subways	$3 586 002	.....	11.34%	3.68%	15.02%	Am. Soc. C. E. Valuation Com. Report, 1913, p. 24.
	East Boston tunnels	2 894 595	.....	6.62%	5.57%	12.19%	
	Boston tunnel and subway	7 623 207	.....	5.48%	3.58%	9.06%	
	Cambridge connection	1 199 904	.....	8.05%	5.20%	13.25%	
		$15 303 708	Average	7.28%	3.80%	11.09%	
	Pennsylvania Railroad tunnels, East River	.....	.....	6.1%	.....	.....	
1885	Washington Bridge over Harlem River, New York City	$2 648 785	1.5%	6.1%	.....	.....	Description by Hutton.
	Pacific Gas and Electric Company. Rate suit data (San Francisco, Cal.).						
	(*a*) Big Creek	$11 339 308	4.76%	6.23%	3.17%	14.16%*	Int. 8.86% making combined overhead and interest exclusive of taxes and contingencies 23.02%
	(*b*) Big Bend, permanent dams	522 437	8.08%	9.30%	1.70%	19.08%*	Int. 3.68% do. 22.76%
	(*c*) Big Bend extension	476 928	5.23%	6.40%	1.83%	13.46%*	" 7.10% do. 20.56%
	(*d*) Kaweah No. 3	511 560	2.88%	7.06%	4.35%	14.29%*	" 7.38% do. 21.67%
	(*e*) Kern Canyon	340 685	1.11%	12.33%	0.14%	13.68%*	" 7.21% do. 20.79%
	(*f*) San Joaquin	1 563 485	1.00%	9.5%	2.0%	12.50%*	" 6.0% do. 18.50%
	(*g*) Tule River	649 096	2.89%	11.63%	1.76%	16.28%*	" 9.96% do. 24.24%
	(*h*) Spaulding drum development	3 498 894	4.30%	6.0%	1.40%	11.70%*	" 6.8% do. 18.50%

* G. P. Cutten, Esq., Counsel to the Company, who furnished these records, writes (August 14th, 1915): "It must be borne in mind that the construction costs in this table are actual costs taken from the books, and all contingencies, etc., which occurred on the jobs are in the unit costs. An evaluator must make separate allowance for these contingencies in his overhead, which would increase the overhead on a comparative basis from 5 to 10 per cent."

(*d*).—Overhead and Interest During Construction on Four Large Undertakings.

Summary of Analysis made by Allen Hazen, M. Am. Soc. C. E.

No.	City and works.	Amount.	Period.	Land only.	Structures only.
1	Board of Water Supply, New York City.	$135 000 000	1905–1915	.....	.....
	Engineering and administration.....	.....	.....	43.08%	14.56%
	Interest at 4.30%, and taxes during construction	.....	.....	37.15%	18.02%
	Total overhead.....	.....	.....	96.23%	35.20%
2	Metropolitan Water Board, Boston.....	.....	.....	.....	.....
	Engineering and administration.....	.....	.....	.....	.....
	Interest at 3.10% during construction.....	.....	.....	.....	.....
	Total overhead.....	.....	.....	.....	†
3	Cincinnati Water-Works Improvements.	$11 000 000	1897–1908	.....	.....
	Engineering and administration.....	.....	.....	.....	12.72%*
	Interest at 3.75% during construction.....	.....	.....	.....	15.24%
	Total overhead.....	.....	.....	.....	29.90%
4	Little River Works, Springfield, Mass...	$2 000 000	1907–1908	.....	.....
	Engineering and administration.....	.....	.....	.....	14.30%
	Interest at 3.5% during construction, about	.....	.....	.....	3.50%
	Total overhead, about.....	.....	.....	.....	18.3%

* Calculated on structures and land taken together.

† See revised analysis by Metcalf and Sherman, of Metropolitan Water Board records, furnished by F. P. Stearns.

Overhead on Structures with Engineering and Administration as it Actually was, and with Interest Calculated at Actual Rate and at a 6% Rate.

No.		Figures in Percentages. New York Board of Water Supply, structures.	Cincinnati Water-Works, structures and land.	Little River Water-Works, Springfield, Mass., structures only.	New York Board of Water Supply, land.
1	Engineering and administration.....	14.56%	12.72%	14.30 %	43.08%
2	Interest rate.....	4.30%	3.75%	3.50%	4.30%
3	Total actual interest.....	18.02%	15.24%	3.50%	37.15%*
4	Average number of years for which interest was paid.....	4.19	4.07	1.00	7.85
5	Interest for the same period at 6%, obtained by multiplying actual interest by ratio of 6% to rate paid...	25.15%	24.39%	6.00%	47.00%
6	Corresponding total overhead.....	43.37%	40.20%	21.16%	110.3%

* Includes taxes.

(*e*).—Interest During Construction on Various Works.

Summary of Analyses by Leonard Metcalf, M. Am. Soc. C. E.

No.	Locality and character of works.	Period of years.	Date of completion.	Reported total cost.	Accrued Interest during Construction to Completion of Structure with Compound Interest at: Assumed actual rate of:	Actual rate.	5%	6%	Remarks.
1	Board of Water Supply, New York City. Reservoirs, dams, aqueducts, etc.								
	(*a*) Water-works structures only	12	1915	$132 760 260	4.30%	18.02%	25.15%	.....	Annual Reports.
	(*b*) On land	10	1915	21 196 700	4.30	37.15	47.0	.....	Allen Hazen, M. Am. Soc. C. E.
					4.30	31.4	.....	.....	Excluding in rate the interest upon award.
					6				Including do.
2	Contract No. 80. Grade and pressure tunnel	4	1913	420 465	4 ± ?	9.0	11.3	13.7%	
3	Contract No. 45. Cut and cover aqueduct	5	1913	1 592 054	4 ± ?	8.8	11.1	13.5	
4	Contract No. 12. Rondout Siphon and tunnel	6	1913	6 515 444	4 ± ?	14.7	18.7	22.8	
5	Contract No. 20. (4.8 miles.) Wooden pressure tunnel	5	1913	3 344 497	4 ± ?	10.5	13.3	16.2	Excluding land, engineering, administration, and interest.
6	Ashokan Main Dams	9	1914	11 773 216	.....	16.5	20.5	25.7	do.
7	Massachusetts Metropolitan Water-Works. (Boston.)‡			$25 317 000					
8	Wachusett Aqueduct	2	1897	1 718 405*	3.1†	3.2*	4.4*	5.1*	
9	Wachusett Dam	10	1904	2 271 000	3.1	8.0	12.0	14.4	
10	Weston Aqueduct	.....	1903	2 788 523*	3.1	5.1*	7.4*	8.7*	
11	Hopkinton Reservoir	4	1893	910 000	4.0	9.1	11.3	13.4	
12	Massachusetts Metropolitan Sewerage Contract No. 66: 5 300 ft. rock tunnel, 9 ft. 3 in. by 10 ft. 2 in.	4	1903	252 861	.....	3.7	6.1	7.3	
13	Boston Transit Commission. East Boston Tunnel	7	1904	2 571 185	3½ ±	9.8	13.5	16.4	

* Includes 1% allowance for lost interest on cash balances and difference between interest paid and that received on reinvestments. F. P. Stearns thinks this an over-estimate and that it may have been as low as 0.5 per cent.

† Interest to date of active use of structure (before completion).

‡ It is to be noted that on the Metropolitan Water Board an appropriation of $27 000 000 was made available in the Metropolitan Water Act at the inception of the work in 1895. This, coupled with the State Treasurer's ability to keep most of the idle funds of the State invested in temporary loans to municipalities within the State at practically equal rates of interest, made it possible for the Board to sell its bonds virtually as the demand for construction funds accrued, without the loss in interest usually resulting from having to sell the securities at one time, and to reinvest the surplus funds in readily convertible loans at the best possible interest rate—practically always lower than the rate obtained on the bonds, until needed. Mr. Frederic P. Stearns states that, as a result of these conditions, the interest during construction was extremely low—probably without parallel.

(*e*).—Interest During Construction.—(*Continued*).

Summary of Analyses by Leonard Metcalf, M. Am. Soc. C. E.

No.	Locality and character of works (structures only), except otherwise noted.	Period of years.	Date of completion, December 31st.	Reported total cost, when put into service, excluding interest during construction.	Accrued Interest During Construction to Completion of Structure, with Interest Compounded Annually at: Approx. actual rate of:	Actual rate.	5%	6%	Remarks.
27	Cambridge, Mass., Water-works.								
	a. Stony Brook Supply.....	4	1887	$538 757	3.75%	7.1%	9.6%	11.6%	
	b. Payson Park Reservoir..	5	1897	685 151	3.75%	7.6%	10.0%	12.2%	
	c. Hobbs Brook Reservoir..	4	1897	993 244	3.75%	5.4%	7.1%	8.5%	
	Totals and Averages			$2 217 132	3.75%	6.5%	8.5%	10.4%	
28	New Bedford, Mass., Lake Quittacus Water-works.....	7	1899	1 256 688	3.66%	9.2%	12.7%	15.4%	
29	Portland, Me., Water District New 14.3 mile 42-in. cast-iron pipe line from Sebago Lake Basin to City.....	4	1912	811 201	4.0%	4.1%	5.1%	6.2%	
30	Springfield, Mass., Little River Supply. Storage reservoir dam, intake reservoir tunnel, slow sand filters, steel pipe line and distributing reservoir....	3	1909	1 455 684	3.5%	2.8%	4.1%	4.9%*	
31	Metropolitan Water-works † (Boston) including land and damages.....	Various periods	1895 to 1907	$23 600 000 $24 653 000	3.10% 3.16%	$0.42% $8.3%	....	....	Allen Hazen. Leonard Metcalf and Charles W. Sherman.
32	New Orleans Water-works.....	6	1909	7 518 868	4.0%	8.3%	10.6%	12.7%	
33	Waterville, Maine, *et al.* Kennebec Water District—New Supply.....	1	1905	252 154	3.57%	2.1%	2.9%	3.5%	7 months' period. Emergency new supply.
34	Chicago, Ill., Water-works Southwest Land and Lake Tunnel System.....	8	1911	3 249 754	4%	14.1%	17.9%	21.8%	
		8	1911	3 307 285	4%	14.6%	18.5%	22.6%	Incl. $57 530.95 item of cost in 1902.

* Actual period 4 years, excluding preliminary investigations. Emergency case. 1910 payments of $806 457 excluded as plant in service.

† See also Nos. 7 to 11 for different structures.

‡ Excludes acquisition of old works, $14 059 964; administration, $262 606; stock, $70 878.

§ On Metropolitan Water Board work the interest-during-construction has been figured to the dates of actual use of individual structures (before completion).

(*e*).—Interest During Construction.—(*Continued*).

Summary of Analyses by Leonard Metcalf, M. Am. Soc. C. E.

No.	Locality and character of works.	Period of years.	Date of completion, December 31st.	Reported total cost.	Accrued Interest During Construction to Completion of Structure with Compound Interest at:				Remarks.
					Assumed actual rate of:	Actual.	5%	6%	
14	Denver Union Water Company. Cheesman Dam.....	7	1905	$1 229 641	5%	18.7	22.8	27.0	Excluding land and rents. Four years (net time) were consumed in investigation, surveys and planning, correspondence with Government, acquiring rights, etc.; and 6.5 years in construction, during a total period of 12 years from inception to completion.
15	Cincinnati Water-Works: Intake, pumping station, filter plant, conduit, etc.....	11	1907	13 329 348	3.75%	15.8	21.2	26.0%	
16	1905-1914 Los Angeles Aqueduct.....	9	1914	$24 526 868*	4.5%	20% +	(Actual)		*Excluding interest.
17	1914 Pacific Gas and Electric Company. Spaulding Dam..	1 +	1914	4 283 359	.....	6.8%	(Actual)		A rush job.
				Structural Costs only					
18	— Big Creek.....	....	.....	$11 339 308	.....	8.86%	(Actual)		Quoted by Pacific Gas and Electric Company Exhibit 15, Cases 477 and 550, before California Railroad Commission.
19	— Big Bend permanent dams..	....	.....	522 437	.....	3.68	.....	.....	
20	— Big Bend extension	....	.....	476 928	.....	7.10	.....	.....	
21	— Kaweah No. 3.....	....	.....	511 560	.....	7.38	.....	.....	
22	— Kern Canyon.....	....	.....	340 685	.....	7.21	.....	.....	
23	— Los Angeles Aqueduct.....	....	.....	18 840 344	.....	19.40	.....	.....	
24	— San Joaquin.....	....	.....	1 563 485	.....	6.0	.....	.....	
25	— Tule River.....	....	.....	649 096	.....	9.96	.....	.....	
26	— Spaulding drum development	....	.....	3 498 894	.....	6.8	.....	.....	

(*f*).—Engineering on Railways.

From Various Sources.

District.	Miles.	Total cost of construction.	Total cost per mile.	Total cost of engineering.	Cost of engineering per mile of road.	Engineering. Percentage of road.

(1).—Actual Cost. Milwaukee, Sparta and Northwestern Railway (Chicago and Northwestern) 1910.

Milwaukee to Clyman Junction	52.97	$5 412 246.85	$102 175.70	$108 428.21	$2 046.97	2.0
Butler Yard	.....	891 707.04	.....	7 654.47	.....	0.86
Clyman to Necedah	89.44	5 396 907.95	60 342.22	165 008.89	1 844.91	3.06
Friendship Yard	.....	364 559.21	.....	4 410.19	.....	1.21
Wyeville to Sparta	28.02	1 430 448.99	62 189.40	46 721.13	2 029.59	3.27
Wyeville Yard	.....	339 005.89	.....	4 581.87	.....	1.38
Necedah to Wyeville (Reconstruction of old road)	13.05	238 186.32	18 251.82	3 535.18	270.89	1.49
Average cost, entire line	165.48	$12 701 161.94 13 834 925.93 Cost, including right of way.	$76 775.65 83 630.09	$336 804.76	$2 034.72	2.65 2.44

(2).—Cases Presented to Interstate Commerce Commission in Argument and Brief, September 1st, 1915.

Name.	State.	Miles.	Percentage.
Kansas City Southern	Missouri	3.4	2.6
Chicago and Northwestern	Wisconsin	178.0	2.8
Illinois Central	Kentucky	82.0	1.3
Sunset Central	Texas	47.0	2.8
Union Pacific	Nebraska	26.0	2.5
Southern Pacific	California	143.0	2.5

"The total investment for these 4 390 miles of western lines was $126 711 214, exclusive of land, equipment, and overhead charge; the charge to engineering was $5 193 116, the construction cost per mile being less than $29 000.

"The East reports 42 cases (exclusive of the Pennsylvania Tunnel and Terminal Company and the New York Central developments in New York). The mileage covered is 817 miles, or but 19% of the length reported by the West, the total investment, excluding land, equipment, and overhead, being $132 528 399, or $5 817 185 more than the 4 390 miles reported by the western lines.

"The engineering cost for these 817 miles was $7 677 089, or 5.79%, an increase in rate of 41% over the western lines. The average cost per mile was $162 239, or 5.6 times the amount per mile expended on the western roads reporting."

Name.	State.	Miles.	Percentage.
Boston and Maine	New Hampshire	7.5	1.7
Pennsylvania Lines West		6.0	2.2
Western Maryland		86.0	2.6
Delaware and Hudson		14.0	2.2
New York, Ontario and Western		54.0	2.4
New York Central		11.0	2.0
Chicago and Eastern Illinois	Illinois	62.0	1.8
Chicago and Eastern Illinois	Illinois	63.0	2.6

"The New York Central and Hudson River Railroad Company, the Barge Canal, and five cities spent $256 800 000 (including land), of which $133 000 000 was expended outside the City of New York. The engineering expense on this work that was charged amounted to 7.6% on all items of construction, including land.

"The Pennsylvania Tunnel and Terminal Company, with 11.81 miles of first track and 69.42 miles in all, cost $111 701 889, exclusive of equipment, taxes, interest, or overhead charges other than engineering; of which amount, $20 679 950 was for land, or 18.5% of the total. The charged cost of engineering was 7.2% of the cost, excluding land and equipment.

"The South reports 30 cases, covering 1 638 miles, with an investment, including land, but excluding equipment and overhead, of $78 546 665. The charged cost of engineering on these 1 638 miles was $3 492 248, or 4.4%, including land."

(*g*).—Land.—Overhead Charges.—Statements Showing Some Expenses of Railroad Companies in the Acquisition of Land.—(*Continued*).

Owning company.	Acquired for:	Total purchase price, exclusive of expense of acquisition.	Expense of acquisition.	Percentage of purchase price.	Remarks.
	Brought forward	$1 622 863	$95 861		
Delaware, Lackawanna and Western Railroad	Cut-off, Pt. Morris, N. Y., to Slateford, Pa	237 566	43 172	18.17	
Hastings and Northwestern Railroad (Union Pacific Railway Company)	New line, Nebr	211 862	9 493	4.48	
Lehigh Valley Railroad Co	Irvington Branch	129 945	2 676	2.06	
	Lehigh and Lake Branch	1 290 588	35 718	2.77	
	Hays Creek Branch	25 163	3 113	12.37	Of the percentage of purchase price, 7.25 represents services of attorneys.
	Seneca Falls Extension	39 993	1 243	3.12	
Long Island Railroad Company	Elimination of grade crossing at Bushwick Junction, Long Island	84 475	3 384	4.01	
	Elimination of grade crossing at Queens, Long Island	73 083	4 067	5.56	
	Additional right of way and station grounds, Brooklyn	712 812	38 473	5.4	Expense ratio as to property purchased, 4½%; condemned, 24 per cent.
	Passenger coach yard, Jamaica, Long Island	234 356	13 371	5.7	Expense ratio as to property condemned, 4.2%; condemned, 15 per cent.
Marion and Southern Railroad (Seaboard Air Line)	Extension in South Carolina	12 856	701	5.45	
Milwaukee, St. Paul and Sault Ste. Marie Railway	Fordville-Drake Line, N. Dak	237 942	27 403	11.51	High expense due to the great number of small purchases—about 160—averaging less than $250. Legal expense was more than 80% of total.
	Ceylon-Downing cut-off	54 248	5 494	10.13	
Norfolk and Western Railway	Main line widening, Nottaway Co., Va	35 775	13 761	38.40	
	Main line widening, Dinwiddie Co., Va	8 483	1 154	13.16	
	Main line widening, Isle of Wight Co., Va	2 529	687	27.18	
Northern Pacific	Passenger coach yard, St. Paul, Minn	411 508	3 553	0.86	Amount paid for improvements $317 437.
	Bear Creek Branch, Mont	114 758	5 384	4.70	
	Knife River Branch, N. Dak	44 566	9 259	20.77	
	New passenger line, Minneapolis	490 023	28 192	5.80	
New York, New Haven and Hartford Railroad	Elimination of grade crossings, Boston, Mass	120 410	4 568	3.70	Includes expenses of trial attorneys, but not of purchasing, title, or general office, which were not separately charged.
	Elimination of grade crossings, Olneyville, Providence, R. I	126 106	4 070	3.20	
	Totals carried forward	$6 321 910	$354 797		

(*g*).—Land.—Overhead Charges.—Statements Showing Some Expenses of Railroad Companies in the Acquisition of Land.

Statement of Thomas W. Hulme, General Secretary, Presidents' Conference Committee, at Conference between Division of Valuation, Members of State Commissions, and Representatives of the Presidents' Conference Committee of the Railroads, May 28th, 1915.

Owning company.	Acquired for:	Total purchase price, exclusive of expense of acquisition.	Expense of acquisition.	Percentage of purchase price.	Remarks.
Baltimore and Ohio Railroad	Produce yard, Baltimore, Md.	$206 724	$10 186	4.90%	Expense ratio as to property purchased, 3½ per cent. Expense ratio as to property condemned, 16.7 per cent.
Bessemer and Lake Erie Railroad	Kremis to Osgood, Pa. (K. O. Line)	63 211	3 299	5.22	$342.96 included in purchase price would more properly be included in expense; and *vice versa* as to $892.56.
Boston and Maine Railroad	Mechanicsville, N. Y., transfer yard	28 185	336	1.30	To cover salaries, $500 has been used.
	Newport, Vt., freight yard	18 450	812	4.40	
	Salem, Mass., freight yard	6 920	554	8.0	
	Guyandotte Valley Extension, W. Va.	28 400	1 848	6.51	
Chesapeake and Ohio Railroad	Buffalo Creek Branch, W. Va.	44 095	1 925	4.37	
	Big Sandy Branch Extension, Ky.	236 953	25 091	10.59	Acquired in 1902.
	Coal River Branch, W. Va.	43 931	2 355	5.36	
	Yard at Peru, Ind.	11 850	964	8.14	Much negotiation required.
	Gravel pit near Peru, Ind.	9 970	612	6.14	
	Terminal purposes at East End, Binghamton, N. Y.	282 129	4 775	1.69	
Delaware and Hudson	Delanson to Schenectady, N. Y.	212 195	32 498	15.25	Of this ratio 12.46% represents expenses incurred in condemnation proceedings.
	Terminal purposes at Schenectady, N. Y.	346 750	8 896	2.56	
	Yard and cold storage plant, Glenville, N. Y.	83 100	1 710	2.06	
	Totals carried forward	$1 622 863	$95 861		

(*g*).—Land.—Overhead Charges.—Statements Showing Some Expenses of Railroad Companies in the Acquisition of Land.—(*Continued*).

Owning company.	Acquired for:	Total purchase price, exclusive of expense of acquisition.	Expense of acquisition.	Percentage of purchase price.	Remarks.
	Brought forward	$6 321 910	$354 797		
Pennsylvania Railroad	Darby Creek Line	1 522 400	73 610	4.84	Of the 4.84% representing expense of acquisition, 1% engineering expense on account of surveys on present and former locations.
Philadelphia, Baltimore and Washington Railroad (Pennsylvania Railroad Company.)	Glynden to Lovell	62 896	6 628	10.53	The large number of properties, and widely scattered ownership, made it necessary to consume much time in traveling.
	Additional tracks, Newport to Pencader, Del	32 205	3 687	11.45	Difficult negotiations and widely scattered ownership, taken in connection with the small total purchase price, made the expense ratio rather high.
Pomeroy Belt Railway (Hocking Valley Railway Company.)	Extension near Pomeroy, Ohio	14 317	926	6.47	
Quemahoning Branch Railroad (Baltimore and Ohio Railroad Company.)	Husband line	56 750	3 464	6.10	In the purchase price, as given, are included $4 859 paid for damages to crops, change of roads, privilege of wasting material, etc. If this amount be excluded from the purchase price, the expense ratio will be 6.60 per cent.
Vandalia (Pennsylvania Railroad West.)	Harmony to Terre Haute, Ind	112 756	4 390	3.89	
	Eagle Creek Connection, Ind	26 013	1 596	6.13	
Winston-Salem South Bound Railway Company (Atlantic Coast Line.)	Tar Branch Terminal Line	240 483	3 558	1.48	The figures submitted show an expense ratio of 1.73%, based on a total purchase price of $205 299. The accompanying statement, however, shows that the purchase price should be $240 483. Hence the ratio of 1.48 per cent.
	Totals	$8 889 730	$452 656	5.4%	

(*h*).—Analysis of Certain Land Purchasers by Railroads.

Details of passenger coach yard purchase, St. Paul.

There were no condemnations, all tracts being bought at private sale.

There were no severance damages, the entire tract being bought in each case.

The law requires the Tax Commission to determine ratio of assumed valuation to real value, and the "assessment method" is based on actual assessment affected by this ratio.

	Before the identity of the purchaser was known.	After the identity of the purchaser was known.
Area purchased	581 120 sq. ft.	388 931 sq. ft.
Total purchase price	$94 000.00	$269 700
Deduct amount paid for improvements	11 083.33	69 132
Amount paid for land	$82 916.67	$200 568
Price per square foot paid for naked land	0.143	0.558
Total naked land value determined by assessment method	44 535.00	49 536
Naked land value, assessment method, per square foot	0.077	0.138
Excess paid over value by the assessment method	$38 381.67	$151 032

Summarized, the above figures show:

Naked land value determined by assessment method.	$94 071.00	23%
Paid for improvements	128 023.47	31%
Paid, in excess of value, by assessment method	189 413.66	46%
Paid for severance damages	0	0
Expenses of acquisition	$411 508.13	100%

The actual expense of the Right-of-Way Department was $3 553.20, or only 0.86% of the total purchase price. There were 21 parcels acquired and two others secured by exchange of site and removal of buildings, making 23 owners dealt with. Expense per parcel, $154.50.

Details of Knife River Branch, N. Dak., Northern Pacific Railway.

	Mercer Co., N. Dak.	Dunn Co., N. Dak.
Valuation of naked land, made by real estate men and bankers, not by employees. Per acre	$22.07	$22.13
Total	$15 709.42	$12 065.12
Total number of deeds	86	63
Total acres conveyed	784.97	615.55
Average acres per transfer	9.12	9.77
Total consideration	27 460.	17 106.
Average consideration per acre	35.00	27.80
Total number of considerations	4	None
Acres condemned	70.86	.....
Award in condemnation cases	1 966.52	.....
Average per acre, award	27.70	.....

Summary.—Both Counties.

Total naked land value	$27 774.54	62.32%
All other elements of value	16 791.89	37.68%
Total purchase price	$44 566.43	100%
Expense of acquisition	9 258.79	
Ratio of expense of acquisition to purchase price	20.77	

Expenses of Acquisition.

	Total.	Average per deed.	Average per acre.
Services and expenses of right-of-way agents	$6 790.75	$46.19	$4.85
Services and expenses of attorneys	705.69	4.80	0.50
Cost of abstracts and opinions of title	1 227.80	8.35	0.93
Cost of recording deeds	329.55	2.24	0.23
Automobile hire	205.00	1.40	0.15
	$9 258.79	$62.98	$6.66

This case affords an illustration of the extremely favorable cost of acquisition. The line was desired by the people. There was but little contest. Two of the condemnation cases were to acquire land owned by the State of North Dakota, the other two were against land companies. Every individual owner was settled with, yet, in spite of these facts, the land cost was $27 774.54, and "other elements" plus cost of acquiring amounted to $26 050.68, or 93.8% of the bare land price.

Details of Bear Creek Branch, Northern Pacific Railway.

The Bear Creek Branch, built in an irrigated valley under high cultivation, paralleling within 2 miles an existing road, was not desired by the farmers.

The assessed value, based on sales of 5 916 acres in 1913-14, was 25.3% of sales. On this the assessor's statement that 25% is a fair average is accepted for part of the district, but 20% was used on part.

Condemnation occurred in seven cases, in each case the award was divided into (*a*) actual value of strip taken, (*b*) damages to remainder.

Area taken by condemnation	23.57 acres	
Award for strips taken	$3 624.00	13.2%
Award for damages	23 930.00	86.8%
Total award	$27 554.00	100%

It is to be noted that the award for strips averaged $153.75 per acre, as against an actual land value, based on assessment method and opinion, of $59.85 per acre plus $20.00 for growing crops. This award was $73.90 in excess of actual value.

The total purchases were as follows:

Bridger to Belfry	155.78 acres at $59.85	$9 323.43	
Belfry to Bear Creek	104.26 " " 25.15	2 622.06	
	260.04 "		
Bear Creek town lots		2 206.08	
Total naked land value		$14 151.65	12.33%
All other elements of value		100 606.60	87.67%
		$114 758.28	100.00%
Expense of acquisition		$5 384.35	
Ratio of expense of acquisition to total purchase price		4.7%	

Details of costs of acquisition, 56 parcels, 260 acres plus 59 town lots.

	Total.	Per parcel.	Per acre.*
Abstracts of title	$725.25	$12.95	$2.70
Recording deeds	111.40	2.00	0.41
Legal	168.25	3.00	0.62
Salaries and expenses right-of-way men	4 379.45	78.20	16.22
	$5 384.35	$96.15	$19.95

Details of Land Purchases, Norfolk and Western Railway, Nottoway Co., Va. Main Line and Second Track Construction.

	Acres.	Parcels.	Average acres per parcel.	Total considera-tion.	Naked land value.	Other elements of value.
Town land	13.40	...	.....	$14 006	$5 870	$8 136
Country land	87.86	...	.....	21 769	10 068	11 701
	101.26	163	0.633	$35 775	$15 938	$19 837

Averages: Town land, average per acre, naked land	$438.06	41.9%
" " " other considerations	607.16	58.1%
Total	$1 045.22	100%
The expenses of acquisition		$13 761.06
Ratio of expense to cost of acquisition		38 4/10%
Legal expense included in cost of acquisition		$11 186.28
Cost of acquisition, other than legal expense		$2 574.78
Ratio of expenses, other than legal, to cost		7 2/10%

* The 59 town lots computed as 10 acres.

Total expense of acquisition, per acre	$135.90
Total expense of acquisition, per parcel	84.40
Expense, exclusive of land, per acre	25.43
Expense, exclusive of land, per parcel	16.10

The elements in "Other elements of value" were $2 105 for moving buildings, $2 040 for severance damages, and the remainder was consequential damages.

	Acres.	Parcels.	Total consideration.	Naked land value.	Other elements of value.
Dinwiddie County	22.54	23	$8 482 66	$3 245	$5 238
Isle of Wight County	5.436	12	2 529	580	1 949

Expense of acquisition, Dinwiddie Co., or 13.16% of purchase price.	$1 154.33,
Expense of acquisition, Isle of Wight Co., or 27.18% of purchase price.	687.32,
Expense per acre, Dinwiddie Co.	51.00
" " " Isle of Wight Co.	126.44
" " parcel, Dinwiddie Co.	50.19
" " " Isle of Wight Co.	52.87

Investigations of the California Commission.

In the proceedings to determine the valuation of the Petaluma and Santa Rosa Railway, the finding was written by Commissioner Shelen. In his discussion of right of way, he states (Decision No. 2348—Case No. 145—Publ. Serv. Reports, 1915 C 752):

"The records of the engineering department contain the analysis of some 1 140 miles of recently constructed railroad rights of way in California, the total cost of which is $8 211 632.65, and the market value of the land for ordinary purposes, at the time of purchase, was $6 293 862.89, which would show a multiple of 1.30. The incidental expense in connection with the acquisition of 844 miles of this right of way was $648 329, which is 9.14% of the amount paid to the grantors. Practically 88% of the costs of lands was for property located within incorporated city limits, and the multiple on this classification was approximately 1.25."

(*i*).—Cost of Acquiring and Assembling Lands for Reservoirs and Rights of Way.

Compiled by Leonard Metcalf, M. Am. Soc. C. E.

Year.	Locality and purpose.	Area, in acres.	Total cost, excluding overhead and interest.	Cost per acre.	Excess of actual cost over appraised value, excluding overhead and interest.	Excess over assessment.	Authority.
1912	Kendall Reservoir, Worcester, Mass., Water-Works	407.84	$3 917*	$95.80*	59%	.....	City Engr. McClure. *Including some buildings.
1915	Fitchburg, Mass., sewage disposal area	115.8	24 323	.....	6%	.....	Engr. Hartwell. One item $7 193 (= 23 8 acres) settled by Court procedure; excess cost, 23 per cent.
1914	Newport, R. I., Water-Works valuation proceedings	.....	.....	.....	118%	.....	Excess of Company's witnesses' estimate based on actual cost, over City's witnesses.
	By condemnation	30.56	.....	712.00	137%	.....	140.72 acres of adjoining land sold short time before for $250 per acre. Fair value estimated, $300.
1899	(Boston) Metropolitan Water-Works in Wachusett Reservoir	4 772.00	1 182 000	248.00	69%	.....	F. P. Stearns, Chf. Engr., Am. Soc. C. E. Val. Com. Rept., December 1st, 1913, p. 56—Appraisers liberal estimate was for land $244 000 plus buildings $453 000; total, $697 000. Owing to compulsory feature the Board added in its estimate 50% excess plus 15% overhead, exclusive of interest. The actual cost slightly exceeded this.
	Individual parcels, included in the above:						
	(*a*) Clinton Settlements excluding mill damages	.....	157 044	.....	54%	97%	
	(*b*) Boylston do	.....	155 439	.....	36%	85%	
	(*c*) West Boylston do	.....	555 910	.....	13.5%	115%	
	(*d*) Sterling Settlements	.....	20 104	.....	7%	141%	
	(*e*) Settlements in Clinton, Boylston, and West Boylston	.....	868 393	.....	23%	106%	
	(*f*) Settlements in Clinton, Boylston, West Boylston, and Sterling	.....	888 497	.....	22.5%	106%	
	(*g*) Right of way acquired for relocation of Mass. Central R. R. (18 parcels)	29.36	20 074	.....	32%	.....	
	(*h*) Right of way for Weston Aqueduct (70)	197.35	110 640	.....	77%	.....	
	(*i*) Ditto (9 parcels)	242.34	60 811	.....	14%	.....	
	(*j*) Court Awards:						
	Central Massachusetts Railroad	1.18	917	.....	116%	.....	
	Weston Aqueduct	0.36	1 147	.....	2 190%	.....	
		3.81	1 785	.....	207%	.....	
		5.64	550	.....	57%	.....	
		17.17	25 270	.....	747%	.....	
		1.96	1 764	.....	320%	.....	
		6.84	9 000	.....	41%	.....	
		5.02	800	.....	16%	.....	
		41.98	$41 233.38	.....	252%	.....	

Wachusett Reservoir Data:

Water surface	4 135 acres
Contents	64 968 000 000 gal.
Shore line, excluding islands	37 miles

Required removal of 6 large mills, 8 school houses, 4 churches, 360 dwelling houses. Old road discontinued, 19.25 miles; new road built, 11.8 miles. Cost of January 1st, 1913:

Wachusett Dam	$2 386 375.23
North Dike	732 264.68
South Dike	137 075.55
Removal of soil	2 536 612.66
Relocation of railroad	881 872.45
Roads and bridges	547 867.76
Real estate taken	3 240 398.41
Damages, business, real estate not taken	532 247.07
Legal and expert, except real estate	8 547.92
	$11 063 261.73

(*i*).—Cost of Acquiring and Assembling Lands for Reservoirs and Rights of Way.—(*Continued.*)
Compiled by Leonard Metcalf, M. Am. Soc. C. E.

Location and purpose.	Area, in acres.	Total cost, excluding overhead and interest.	Cost per acre.	Excess of actual cost over appraised value, excluding overhead and interest.	Authority.
Kensico Reservoir	3 182	Award	$742		All but 2 acres settled for. Area, 1.53 times water surface.
		Expenses, 58.17%.	404		
			$1 146	100%	Hazen estimates fair value of this property during last 5 years at $300 to $1000 per acre.
Hill View Reservoir	163	Award	$8 796		
		Expenses, 44.01%.	3 871		
			$12 667		
Southern Aqueduct	957	Award	$1 987		Includes East View filter site.
		Expenses, 39.66%.	787		
			$2 774		
City Aqueduct (Part owned by City)	166 taken 90 proceedings	Award	$6 585		
		Expenses, 15.03%.	962		
			$7 547		
Springfield, Mass., Water-Works				50%	Hazen estimates excess cost of lands bought at about 50% "a remarkable record and one that will be very seldom equalled in city business."

(*i*).—Cost of Acquiring and Assembling Lands for Reservoirs and Rights of Way.—(*Continued.*)

Compiled by Leonard Metcalf, M. Am. Soc. C. E.

Year.	Locality and purpose.	Area, in acres.	Total cost, excluding overhead and interest.	Cost per acre.	Excess of actual cost over appraised value, excluding overhead and interest.	Authority.
	Fitchburg, Mass., Water-Works, Wachusett Lake	118.88	$8 050	$68	21%	Valued by three appraisers for City.
1905....	Cambridge, Mass., Right of way for Hobbs Brook Conduit	2.34	20 639	9 873	28%	Excluding one parcel, the excess was 65%, including buildings.
	(Boston) Metropolitan Water-works:					
	(*a*) Wachusett Reservoir	7 946	2 959 300*	372*		*Including mill settlements and water diversion damages.
	(*b*) " Water-shed	556	148 890	268		
	(*c*) " Aqueduct	309	81 677	264		
	(*d*) Sudbury Reservoir	2 772	658 994	237		
	(*e*) " Water-shed	391	16 522	42		
	(*f*) Weston Aqueduct	480	188 925	393		
1914....	New York, Catskill Water Supply	21 327	10 669 946	500		Based on Board of Water Supply figures, December 31st, 1914, and estimate of 1905 by J. Waldo Smith, M. Am. Soc. C. E. Litigation on value of 345 acres still pending.
	Interest		1 648 801	78		
	Indirect and consequential damages		294 801	14		
	Fees, legal and sundry expenses		4 188 795†	200		†No. 39. 26% overhead excluding interest. See analysis below. (p. 7 of Hazen's letter.)
			$16 808 344	$801		
	Excluding interest			$723		
	Ashokan Reservoir	15 221	Award	$243		Litigation on 316 acres still pending. Area 1.87 times water surface.
			Expenses, 61.41%.	155		
				$398	100%	A little more than twice Engr. Sears, estimate.
	Northern Aqueduct	1 637	Award	$470		Settlements on all but 37 acres.
			Expenses, 91.08%.	428		
				$898	800% or more.	Hazen estimates value of these lands at between $50 and $100 per acre.

APPENDIX IV.

Records of Abandoned Property.

The Committee calls the attention of engineers and administrators of public utility properties to the desirability of keeping as clear and complete a record as possible of the property abandoned by the corporation, annually and in aggregate amount.

Such records will be of great service to Courts and commissions, in assisting them to determine the annual depreciation allowance which should fairly be made for any public utility property, in the light of its past history, as well as of an intelligent forecast of its probable future.

It is obvious that such records—when used as a check on, or when coupled with, an estimate of the accrued depreciation on the existing property—will reduce substantially the element of uncertainty which must attend all such computations, based, as they must always be, on probable future, as well as on past, conditions.

As an example of such records within the water-works field, and of their applicability, the following table is cited, embodying the results of a continuous effort to collect such data, during a period of 15 years, more or less, on the part of Allen Hazen and Leonard Metcalf, Members, Am. Soc. C. E.

It seems highly probable that, by keeping such records in detail, there will be developed, in the course of time, sufficient information to admit of the setting aside of the fair annual depreciation allowance on a basis of percentage-of-gross-revenue of the operating company, a method which would have much advantage, on account of its simplicity of accounting, as well as for other reasons.

Inasmuch as this table is submitted for suggestive purposes only, and not for direct application, it is unnecessary to record here a description of the various plants, or the methods of determining the figures contained in the record, though it may be added that these figures have been taken in large measure from authoritative sources, such as Court, commission, or agreed, findings.

Depreciation Records of Some Old Water-Works.

Assembled and Analyzed by Allen Hazen and Leonard Metcalf, Members, Am. Soc. C. E., December, 1917.

(1)	Portland, Me., Water District. (2)	Racine, Wis. (3)	Denver Union Water Company, Denver, Colo. (4)	Pennsylvania Water Company, Wilkinsburg, Pa. (5)	Spring Valley Water Company, San Francisco, Cal.§ (6)	Indianapolis Water Company, Indianapolis, Ind.¶ (7)
Date of record	1907	1913	1913	1911	1913	Dec., 1915
Authority	Hazen and Metcalf	Hazen	Metcalf	Metcalf	Hazen and Metcalf	Metcalf
Population	78 200	38 002	242 000	90 000	453 000	270 000
Whole period of operation, in years	39	26	43	23	55	45
Average age of investment, in years	18	16	20	12	26	16**
Gross revenue during whole operating period	$6 605 000	$1 343 000	$22 000 000	$3 923 000	$76 968 000	$13 972 000
Gross reproduction cost of structures	$2 594 000	$1 219 000	$10 411 438	$3 128 389	$25 537 000	$7 906 000
Abandoned structures	137 000	19 000	1 420 505	250 000	3 420 000	883 000
Total (including overhead and interest during construction)	$2 731 000	$1 238 000	$11 831 943	$3 378 389	$24 957 000	$8 789 000
Accrued depreciation, existing structures	$498 000	$70 600	$1 320 970	$271 700	$2 800 000	$736 000
Accrued depreciation, in abandoned structures	137 000	19 100	1 420 505†	250 000	3 420 000	883 000
Total depreciation, in amount	$633 000	$89 700	$2 741 475	$521 700	$6 220 000	$1 619 000
" " in percentage	23.2%	7.24%	23.2%	15.4%	24.9%	18.4%
" " in percentage on average net reproduction cost‡					28.5%	20.3%
Average annual straight-line depreciation, in percentage, based on:						
(*a*) Gross reproduction cost and abandoned structures	1.29%	0.45%	1.16%	1.29%	0.96%‡	1.13%
(*b*) Net or depreciated reproduction‡	1.46%	0.47%	1.31%	1.39%	1.10%§	1.23%
(*c*) Gross revenue during whole operating period	9.57%	6.7%	12.4%	8.00%*	8.1%	11.6%
(*d*) Per capita (total depreciation divided by present population multiplied by average age)	$0.45	$0.15	$0.57	$0.48	$0.53	$0.42

* Figures 7.52%; 8% believed to be a fairer figure, however (Metcalf).

† Including Sullivan Reservoir.

‡ On basis of original cost of structures of $21 306 000, a straight-line depreciation rate of 1.25% was found to yield $6 774 000; depreciation allowance (including abandoned property) as of December 31st, 1913. Note that $\frac{\$21\,306\,000}{28\,957\,000} \times 1.25\% = 1.07\%$, an approximate check.

§ Based on decision of Judge H. M. Wright, Standing Master in Chancery, U. S. District Court, Northern District of California, 1917.

‖ Gross reproduction cost of existing structures plus abandoned structures, minus one half of "Total Depreciation."

¶ Based on decision of Indiana Public Service Commission, March 15th, 1917.

** 14.2 years (approx.) on Sinking-fund basis applied to existing structures.

	Average of 6 plants.	Average of 5 plants, excluding Racine.
	1.05%	1.17%
	1.16%	1.30%
	9.4%	10.0%
	$0.43	$0.47

DISCUSSION

Mr. Willoughby.

J. E. Willoughby,* M. Am. Soc. C. E. (by letter).—In its conclusions on the Fundamental Principles of Valuation, the Committee fails to direct especial attention to the fact that the principles and methods must be such as will insure to a property subject to competition an opportunity to benefit in what is sometimes called the unearned increment, in order to attract capital to the service of the public. The investor in a railroad property must have a hope of gain beyond the current rate of interest, because there can never be a guaranty that a railway (as it is always subject to competition) will earn the current rate of interest on its "fair value".

The Committee's classification of "New Properties", "Old Properties under Continuous Regulation", and "Old Properties not Under Continuous Regulation", leads to the assumption that the Committee considers the fair value of the property to be related in some way to past accounting. Such a conclusion is unfortunate. The fair value of a property is not affected in any way by the accuracy or inaccuracy of the accounting. The book record only, of the cost of a property, is thus affected. Even if the capital expenditure accounting were correctly kept throughout the history of a property, the accounting would show merely the monies expended on it, less retirements, and the accrued depreciation, if any, and the accumulated assets. The accounting will not show the fair value of the property.

The writer is not in sympathy with the idea that a property has one value for rate-making purposes, another value for capitalization, and yet another for taxation. It is quite true that property of all kinds is returned for taxation at less than its market value. Some States—*e. g.*, Alabama—fix by statute what percentage of the market value shall be subject to tax levy; in other States, such a percentage is fixed by common consent. It is also true that it will be imprudent for the operating management of a steam railway property to capitalize many items of expenditure which the Interstate Commerce Commission Accounting Classification permits—*e. g.*, the assessment against the railway for a sewer—because the expenditure, being in the nature of a tax, ought to be provided for out of income. Only those expenditures should be capitalized by the operating management as may increase the revenues of the railway; as may lessen the operating expenses; or as may be conducive in some measure to the safety, comfort, or convenience of the employees or patrons of the railway. The rates fixed by and for the railway should be sufficient to provide for expenditures out of income to cover all expenditures which do not come within this classification.

* Wilmington, N. C.

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The exception which the Committee makes in its list of "Physical Property to be Included in Valuation" (page 1319) under *"Excessive Size or Capacity"* is not consistent. The Committee correctly proposes to value the actual property as existing on the day of valuation, and not a substitute property serving the same purpose. The Committee should not vest the engineer in charge of the valuation with the power of reviewing the errors of judgment of the original construction, because that is but a step toward valuing the theoretical property, which will serve the same purpose, and is abhorrent to the Committee's views. Under the exception, much of the present railway construction will be eliminated in part from a valuation. There are noteworthy examples in the Union Passenger Station at Washington, the Pennsylvania Terminal in New York City; and in abnormal expenditures for low grades and multiple tracks in some recent railway construction.

The writer will add, as a corollary to the paragraph "Title to Property not Conclusive", that all structures on land to which the owner of the public utility has title should be included in the valuation. The inventory of land will necessarily include all lands to which the owner has paper title, and not held adversely by others; and also all lands which the owner holds adversely to others, the owner being without paper title. The theory of the law, that ownership of land covers ownership of all fixed improvements, gives to the railway owning the land all fixed improvements thereon, although those improvements may have been paid for wholly or in part by others. This view is not inconsistent with the theory of cost of reproduction which requires us to consider the railway under investigation as the junior line. Deeds to lands and the possession of lands in absence of deeds are the physical items of an inventory of lands, just as the cross-ties, rails, and fastenings are physical items of an inventory of track. In an estimate of cost of reproduction, all structures and appliances that exist to enable the railway under investigation (called *A*) to cross another railway (called *B*) must necessarily be inventoried in favor of the railway *A*, regardless of ownership of land, because such expenditures are a part of the cost of crossing. The railway *A*, under investigation, being nonexistent, must provide such structures and appliances as will enable railway *A* to surmount the physical obstacle which exists as railway *B*. In like manner, when railway *B* is under investigation, the cost of structures and appliances to cross railway *A* must be inventoried in favor of railway *B*. This double inventory is not contrary to the public interest, because the public benefits enormously in the reduction of unit costs from the assumption of the existence of all railways except the railway under investigation, and suffers only a very small addition to the collective estimate of cost of reproduction of all the railways. The writer has noticed with impatience the recent discussion of methods for dividing the cost of crossing structures between the two railways

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on some historical basis, because such discussion contemplates merely a way to prevent the assessment twice of a very small item of the cost of reproduction, and at the same time to lessen greatly the actual historical costs of the property under investigation in all items that the presence of the railway to be crossed will affect.

The Committee is correct in its conclusion on "Cost of Reproduction" (page 1321) that the identical property is to be reproduced, and not a substitute, and that normal present conditions shall determine the prices and methods for doing the work. It is worth while to review just what these conclusions require for railway valuation:

1.—The rail and fastenings, ties, ballast, and track accessories must be considered as new, and of the quality inventoried. It is true that many branch lines of the greater systems were laid with relay rail on the date of first construction, but such practice was a result of the development of the system. It will be inconvenient to go into this feature of development expense—*i. e.*, to assess the losses to the railway due to the removal of rail from principal lines, before the full life of the rail had been obtained, so that relay rail for branches might be available. Considerable quantities of relay rail are not sold in the market for new construction, and therefore it is inconsistent with the theory of reproduction to estimate relay rails for branch lines when the material is not available for purchase for such works. By assessing the decretion to the rail fastenings and track accessories in branch lines, from new to the date of inventory condition, the interest of the public will be fully protected. Many sections of the country are without suitable ballast, and ballast is necessarily handled for long distances, over tracks that have been surfaced on a temporary ballast. A haul is now being made, on gravel ballast to points in Florida, that averages 490 miles. The unit costs must cover the haul. The construction programme will provide properly for the use of temporary ballast, and the hauling and placing of the permanent ballast after completion of the track laying, but the time limit for construction must be greatly extended, and special equipment must be provided for. It will be improper to consider that the entire energies of the railway will be exerted in hauling ballast. The ballast movement will be subject to the delays of the railway's date of valuation traffic—that traffic being one of the present-day conditions.

2.—The cuts and fills must be of the dimensions and of the materials shown on the date of valuation. A fill made of rock or of clay in a sand country must be so inventoried, regard-

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less of whether the adjoining country exhibits only sand. On many of the railways in the Atlantic Coastal Plain, where the soils are largely sand, clay has been hauled many miles, and at great cost, to aid in making a stable embankment. A cut that, at the date of the inventory, is too narrow for steam shovel operation must have unit prices affixed commensurate with the cost of removal by wagons, carts, scrapers, or hand, and must have a time limit sufficient for these slower methods. The cuts in Florida sand (which is too light to support teams) must be considered as being removed by hand (or by steam shovel if the dimensions warrant), and must have unit prices affixed on such basis. Where present-day construction methods still provide for the two-way basis of pay for earthwork, the two-way basis must be assumed. Embankments through the swamps of the Southeast, which will require trestles for construction, on account of lack of available borrow, must have that feature of cost considered. Especial attention must be given to inventory the condition of the material in the top surface of the roadway. All the older railways, by the tamping action of traffic, and by the provision of select soils for track surfacing in past years, have built up the roadbed into a dense crust, impervious to water, and of great sustaining power for the ballast. This crust is an asset to be inventoried. It is not usually obtained in present-day construction because ballast is put directly on the soft roadbed, with resulting water pockets and other objectionable features. A rock cut in a town or city must have unit prices large enough to cover the extra cost of taking the rock out so as not to injure persons and property adjoining, although the town or city may have been built up about the cut after the latter was actually excavated. Borrow from an embankment must be taken from points now available. The inventory must locate available borrow-pits. Borrow cannot be assumed as being taken from improved city property, although the borrow may have been originally obtained from the grading of vacant lots before the city was built.

3.—The bridges, trestles, and culverts must be of the dimensions and material shown on the date of inventory. Where stone, sand, and gravel for masonry is available in quantities necessary for the completion of the work within the adopted time limit only at great distances from the work, these materials must have unit prices sufficient to cover the cost of hauling from those distant points. Concrete is now being built on a new line in Central Florida, for which the gravel is hauled

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600 miles, and for which sand is hauled 200 miles, because there is neither suitable gravel nor sand for concrete available in the territory through which the new line is being built.

4.—The right of way of the railway to be valued is cleared and usually grubbed. Proper consideration of these items requires that they shall be assessed on the basis of the condition of adjoining lands. If the adjoining lands be fields that are now cleared and grubbed, the assumption lies that the right of way will be cleared and grubbed, and *vice versa*.

5.—Due consideration must be given to the normal resources of the country as to labor and supervision. Labor is limited in amount, and any great increase in demand for labor causes the wages to rise and carries increased unit costs. The construction programme must not be shorter than the normal labor resources warrant.

6.—The annoyances, expenses, and delays of and to the conduct of construction works under present-day public antagonisms, political chicane, and legal proceedings must all be incorporated in the costs of reproduction.

The writer is not in accord with the Committee that "history must also be considered, to determine what is to be reproduced, the conditions under which it is to be reproduced, and how estimates must be made". History has no place in an estimate of the cost of reproduction of a railway, except in so far as history is reflected in the experience of the engineer who makes the valuation. In a railway, the items to be reproduced are capable of being inventoried to the extent that such items are now duly inventoried in the construction engineer's estimate in advance of actual construction of present-day works. The idea underlying cost of reproduction is to fix: For what money can the existing property be built now? To mix the past with the present will only confuse the issue.

In making a proper financial and construction programme, unlimited capital must not be assumed. All our large systems of railways have been built up slowly, and the capital required has been available generally on reasonable terms. The tendency of engineers in fixing a programme for reproduction is to consider the physical items only as affecting the time limit. Experience does not justify ignoring the effect which capital requirements will have on the time limit, the unit prices, and the total cost of the property.

There is a value due to continuity which attaches to a railway right of way, but this value is not to be considered in connection with the cost of reproduction. It must be considered as a part of the fair value of the property when that value is fixed.

Mr. Willoughby.

The writer expresses his approval of the distinction made by the Committee between "decretion" and "depreciation" (page 1324). For a railway, the Committee might have brought out more clearly the fact that decretion is an operating expense which for the older railways is taken care of by annual renewals, and which is invariably more than offset by assets of the railway other than those entering into a cost of reproduction valuation.

The Committee's conclusion on depreciation (page 1328) is a correct one. The words "funds held in reserve for such property", mean, of course, securities as well as cash.

It is well to remember that, in ascertaining the value of accrued depreciation, the basis is not the original cost of the item, but the replacement cost. As an example, the labor cost of track laying and surfacing on new lines varies from $1 000 to $2 000 per mile. The labor cost of relaying the rail and fastenings, chargeable also to track laying and surfacing under the Interstate Commerce Commission Classification, varies from $200 to $500 per mile. It is proper to apply decretion to track laying and surfacing, but only to the sum necessary to relay. In some of the tentative valuations of railways lately issued by the Interstate Commerce Commission, depreciation is applied to the cost of track laying and surfacing as of new lines, and the accrued depreciation so determined is in excess of the total cost of relaying, which means that the decretion is more than 100%, and is absurd.

Appreciation is an asset to be added to cost of reproduction new. The Committee is correct in the conclusion that appreciation is not to be used as an offset to depreciation. This conclusion is true because appreciation, being an item to be inventoried and valued as a part of the cost of reproduction new, cannot be used as an offset, any more than other items of the physical make-ups of the property can be used as such an offset. The solidification, seasoning, and adaptation of a railway is as real an item of construction cost as is the cost of haul of material from cut to fill.

"Development Expenses" (page 1329) should properly include the losses due to keeping the property up to the requirements of public use. Many items of permanent railway construction are abandoned because substitute structures must be built to meet the public needs. As an example, consider the abandonment, by the railways of the Southeast, of pile foundations for wooden trestles when public service forced the substitution of 10-ft. trestle spans for 12 and 13-ft. spans. There are single systems in the Southeast with more than 70 miles of wooden trestles. The foundation piles abandoned as a result of the public requirements will cost not less than $10 000 per mile. The abandoned property, being wood permanently moist, has a life expectancy that is practically perpetual.

Mr. Grunsky.

C. E. GRUNSKY,* M. AM. SOC. C. E. (by letter).—No one who has attempted a solution of the valuation problem as it is presented when the rates of public utilities are to be fixed, will underrate the amount of work done and the careful consideration which has been given by the Committee to the preparation of this report. It represents an earnest effort, in the light of the present state of the art, to lay the foundation for the procedure which should be followed when rates are to be fixed, on the assumption that the decisions of the Courts thus far rendered and as ordinarily interpreted are to be accepted as final. The Committee, however, has failed to recognize the fact that it is unwise to make "value", as called for by the Courts, which results from the earnings, the starting point when rates, and therefore earnings, are to be fixed. The Committee has accepted as reasonable the proposition that there need be no stability to the rate-base, that the rate-base may change with the changing value of the elements of which any public utility plant is composed, and that the determination of this fluctuating, constantly changing, "fair value" is necessary, because the Courts hold that "fair value" is the proper starting point.

The writer has written so much on this subject, and his views are so well known, that he will make his remarks very brief. He feels certain, however, that the method of procedure when rates are to be fixed under which the life of the utility, considered in its entirety, is regarded as unlimited, and under which the use of a definitely ascertainable rate-base (not controlled by value) is possible, has so much in its favor that it will, in time, be generally adopted. This method is not mentioned by name, but is closely related to the method described in the report as the "Replacement Method." The Unlimited-Life Method is, to be sure, a replacement method, but not such as would require the exact amount necessary in any year for replacements to be collected from the rate-payers in that year. Neither is its application limited, as suggested by the Committee, for the Replacement Method. The replacement requirement under the Unlimited-Life Method is to be forecast in a reasonable way, and suitable provision is to be made in the earnings to have the money for making replacements available when required. It is immaterial how the estimate is made; but it is essential that all funds collected for replacement purposes be properly accounted for. If the computation has been at fault, and the replacement fund is found after a time to be inadequate, earnings must be increased. If, on the other hand, there is inordinate accumulation in the replacement fund, then the allowance for the replacement requirement should be reduced. Under the Unlimited-Life Method of procedure the need of estimating the accrued depreciation of the entire plant falls away. The rate-base, once determined, remains unchanged until the plant is extended or modified, or until the public

* San Francisco, Cal.

desires to put itself into the position of part owner by retiring a part of the owner's invested capital. Mr. Grunsky.

The soundness of the theory on which the Unlimited-Life Method of procedure is based seems to have been generally recognized. The simplicity of its application and the advantage which it has of requiring lowest earnings in the early years of operation, will compel its general adoption or re-adoption whenever the Courts shall have found it desirable to modify their recent decisions which heretofore have been interpreted to hold that value must be made the starting point, ignoring, apparently, the fact, to which attention cannot be called too frequently, that value is not the premise for the determination of earnings, but is created by the earnings.

There is nothing which would prevent the Courts from receding from the position apparently taken in the Knoxville case, in which the United States Supreme Court said:

"If, however, a company fails to perform this plain duty and to exact sufficient returns to keep the investment unimpaired, whether this is the result of unwarranted dividends, over issues of securities, or of omission to exact proper prices for the output, the fault is its own."

This position, if it means rates at all times to yield earnings which will include so-called depreciation, is not sound doctrine, because the cases will be rare indeed in which, immediately on the beginning of operations, the earnings can be made adequate to keep the investment unimpaired. Some time must be allowed during which the earnings will fall short of the amount apparently called for by the decision of the Court. This time may be only 1 year, or it may be 5 or 10 years. If the opinion of the Court is regarded as applying with equal force to the years already past as to those yet to come, which seems to follow from the context in the decision, then either a sacrifice may be demanded of the owner, or the few rate-payers of the early years may be required to carry an unwarranted burden.

It is sound doctrine, and reasonable and fair to both the rate-payer and the owner, to estimate the required earnings so that there will be no immediate retirement of invested capital, but only suitable gradually increasing provision for necessary renewals. The renewal or replacement fund, however, can be regarded at any time as a return of capital, when the business is to be closed out, or when the public desires to participate in the ownership. A method of procedure is thus marked out, which is remarkably simple and makes unnecessary a consideration of value as the starting point when rates are to be fixed.

The rate-base thereunder will start with the capital reasonably and properly invested, not swelled by appreciation nor reduced by depreciation. This is substantially what Mr. James T. Shaw contends for

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in a statement presented on November 9th, 1915, to the Railroad Commission of California in the Telephone Rate Case (Application No. 1870), when he asks that the "actual performance" be made the "rate-base." In his argument, he reinforces the plea, which is being made by many engineers and economists, that value be regarded as a "result and not a premise."

Influenced perhaps by an earlier presentation of this subject by Mr. Shaw, the Public Service Commission of the State of Washington, in the proceedings relating to telephone and telegraph rates (Cases Nos. 1810 and 1825), on April 25th, 1916, says:

"The statute is silent upon the question of the finding of 'fair value' or a base for rates. The Commission is directed to find the 'market value', but no one contends that the 'market value' is always a fair basis for rates. Since the Commission is required to ascertain the fair, just, reasonable and sufficient rates for telephone service, the Commission will assume that it is authorized to find a 'rate base.'"

Relating to value as a basis for rate-making, the Commission says:

"If we take this definition of the term 'value' and make such value the basis for rate-making, each time we increase the return we increase the desirableness of the property or properties, and on the other hand if we decrease the return, we decrease that which makes the thing desirable; and so, if we decrease the return we decrease the value, and if we increase the return we increase the value."

* * * * * * *

"To say that rates are to be based upon the value of the property, using the term in its usual and ordinary sense, is to say that rates shall be based upon one premise to-day, another to-morrow. So we must conclude that when the Courts said that rates were to be based upon 'fair value', they could not have meant to use the word 'value' in the sense in which the word is ordinarily used and understood."

Some Courts, too, are beginning to recognize the futility of requiring that "fair value" shall be made the "basis of the calculation." Thus, for example, the Supreme Court of Idaho in a recent decision (1915), Pocatella Water Company case, Murray *vs.* Public Utilities Commission, reversing the Public Utilities Commission of that State, says in reference to depreciation:

"In other words, if it be demonstrated that the plant is in good operating condition and giving as good service as a new plant, then the question of depreciation may be entirely disregarded."

It is the duty of the valuation engineer and the economist to point out, as occasion arises, that it would be error on the part of the Courts to insist that "fair value" be made the basis of the calculation when rates are to be fixed. It may not be easy, however, to secure general admission that other procedures than the one thus far approved are sound. The Courts must act with caution, and must be convinced

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beyond peradventure before taking a new stand. The writer believes, however, that the decisions which seem to determine what should be made the "rate-base" are not so final or explicit as to be removed from attack, or at least from possible new interpretation. The decisions of the Courts which make value the starting point, have been rendered to prevent confiscation of property and to protect rate-payers from inordinate exactions. The method supposed to be the only one acceptable to the Courts is illogical and difficult, and is practically impossible of satisfactory application, and yet there is available a simple, logical, and perfectly fair method of procedure, the advantages of which will be generally admitted whenever its application shall have obtained the sanction of the Courts.

Heretofore, the writer has called the attention of the Profession to the fact that the ordinary application of sinking-fund methods to compute the amounts which should go into a depreciation, or into a replacement, fund are materially in error.* It is self-evident that the actual term of usefulness of any article will never agree exactly with the term which was predicted for it when it went into use on the basis of probable life for articles of its class. When consideration is given to the departure of the actual term of service of individual articles from their probable life or expectancy, it will be found that the computation of the annual depreciation increment or of the annual replacement requirement is quite different from that computed in the ordinary way from their probable life.

It is not proposed to present conclusions on this subject, but only to express regret that the Committee did not take the time to ascertain, approximately at least, the magnitude of the effect of the departure of the actual useful life of articles from their expectancies.† The report is silent on this subject, and, being silent, only reflects the present unsatisfactory state of the art. Any one attempting to apply any method of procedure which involves a deduction of depreciation will find difficulty in making provision in the accounting for the individualized articles which go out of use before the end of their predicted term of usefulness, as well as for those which serve beyond this originally allotted term of usefulness. He will find that the Compound-Interest Method is not, as was originally claimed, an equal-annual-payment method, even when interest rates are uniform, that the Sinking-Fund Method might as well be discarded for an approximation method similar to the Straight-Line Method, and that the Straight-Line Method imposes an unwarranted burden on the rate-payers of the early years, which is undesirable and unnecessary. Any "present-value" method, as already stated, must be accompanied by

* *Transactions*, Am. Soc. C. E., Vol. LXXV, p. 837; also, Vol. LXXIX, p. 761.

† See "Valuation, Depreciation, and the Rate-Base", by C. E. Grunsky, pp. 104 to 122.

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frequent re-determinations of value. Under all the methods described by the Committee, except the Replacement Method, bookkeeping will be a complicated art.

The application of the Compound-Interest Method of procedure is by no means as simple as the Committee assumes when, on page 1474, it says:

"There is no ambiguity in the foregoing described method; no chance for confusion; the method is exactly analogous to that described under the straight-line theory of depreciation."

This would only then be true if all individualized articles would actually go out of use at the ends of their respective probable life terms.

The Committee has not offered a satisfactory solution of a vexed question. It leaves the whole matter in confusion. Nevertheless, the facts presented in the report and the academic discussion of the results that would be obtained under various methods of procedure, on the assumption of certain premises, not to be realized in practice, is a valuable contribution to the literature on valuation and rate regulation, and will be of material aid in hastening the acceptance of fundamental principles and the general adoption of a satisfactory method of procedure.

The fact that the profit to which the owner of a public utility is entitled may be expressed in terms other than as an increment of the rate of return applied to a rate-base, has not been brought out in the Committee's report, and yet this deserves discussion, as it smooths the way for determining reasonable rates without starting with "value." The purpose of this extension of the writer's discussion* is to make this fact clear. The Courts are compelled to determine whether or not the allowed earnings are confiscatory. Consequently, they have concerned themselves with value, regardless of how it may have been built up. The rate-regulating authority, on the other hand, should be and is free to use any proceeding which to it appears to be convenient and equitable in determining what the earnings and, therefore, the rates should be. This fact does not seem to have been recognized or accepted by many of the public service commissions, and they, therefore, following the lead of the Courts, are nearly all concerned with the value of public utility properties and find themselves forced, in some cases, as the writer has instanced in his preceding discussion, to read new meanings into the word "value." If every property to be valued was new, there would be comparatively little difficulty in estimating the amount that should be assumed to be reasonably and properly invested therein, but there is serious difficulty when a property is no longer new. The service rendered by, or the output

* This is an addition to the previous part of Mr. Grunsky's discussion.

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of, a property which is mature and well-seasoned, is ordinarily better than, or at least as good as, the same service or output of a new plant. From the standpoint of the value of the service rendered, therefore, the rates determined for a new plant should be appropriate, too, for a plant the parts of which may show all manner of accrued depreciation. Except for the requirement apparently laid down by the Court that "value", interpreted to mean "present" or "depreciated" value, is to be taken into account, there is, in other words, no need at all of considering "accrued depreciation" (not to be confounded with "deferred maintenance") when rates are to be fixed. Consequently, as already stated, the principle of starting with a rate-base undiminished by depreciation is fundamentally sound.

When thus determined, however, something more than the rate-base is to be brought into consideration. The rate-base may express what the sacrifice was, which the owner had made and on which (to cover interest and expenses incident to securing money for the enterprise) he should receive an adequate interest return. In addition to an adequate interest return on such a rate-base, however, he may reasonably expect, and certainly is entitled to, compensation for management, and for business hazards; and he is entitled, also, to a participation in the prosperity of the community which he serves. In dealing with "value", it has become customary to include therein some allowance for "going value" and also to fix the rate of return on the resulting "fair value" higher than ordinary interest rates. A way has thus been found to permit the utility to earn more than interest on the investment. The business yields a profit. Unfortunately, it is difficult, if not impossible, to establish any fair or acceptable rule for determining the fair rate of return which ordinarily is to include the profit item. That this is difficult is due to the fact that one utility is of a character involving a large investment compared with the annual gross income, and another shows the reverse condition; a large amount of business is done on a small investment. Water-works may be suggested as of the first class, particularly when the areas owned for reservoir and water-shed purposes are extensive. Express companies suggest themselves as belonging in the second class. Their ownership of property and capital investment may be but nominal, although their annual receipts are large.

Why not give consideration, therefore, to the volume of business, to the annual gross income, as well as to a rate-base, when determining what the earnings should be? In other words, let the profit be brought into a fair relation to the volume of business, instead of attempting to predicate it on the rate-base, or that uncertain element, "value", which is, in part, created by the profit.

. When an interest return is allowed to the utility, which will cover the cost of money borrowed for legitimate enterprises of the character

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of such utility, and this interest rate is applied to a rate-base estimated from legitimate proper investment undiminished by accrued depreciation, certain profit allowances, about as noted subsequently, will be found to be appropriate. It is assumed that, in determining the cost of borrowed money, due allowance will be made for discounts and commissions. The profit allowances, if made as suggested, would represent compensation in lieu of current appreciation and interest on the going value, or other elements of value not included in the rate-base. In the last analysis, however, the capitalization of this profit allowance would show that intangible values have actually been created and are demonstrable. The reversal of the usual procedure appears to be advisable. Instead of starting with the intangible values which are to be created by the earnings, let the profit be fixed, and the intangible values will take care of themselves. The suggested profit allowances are as follows:

When	gross	income	is	$10 000	about	15%	$1 500	per year.
"	"	"	"	100 000	"	14.5%	14 500	" "
"	"	"	"	500 000	"	13.5%	67 500	" "
"	"	"	"	1 000 000	"	12.5%	125 000	" "
"	"	"	"	3 000 000	"	10%	300 000	" "
"	"	"	"	5 000 000	"	9%	450 000	" "
"	"	"	"	10 000 000	"	8%	800 000	" "
"	"	"	"	20 000 000	"	7%	1 400 000	" "
"	"	"	"	40 000 000	"	6%	2 400 000	" "
"	"	"	"	100 000 000	"	5%	5 000 000	" "

The allowances here suggested as additions to the interest return on the rate-base are only tentative, and remain subject to modification on further study. Furthermore, no such general suggestion as is here made could be expected to fit every case. These allowances, moreover, are intended to apply under ordinary conditions, and do not cover the larger return which the owner has a right to expect under the unusual condition when, by introducing a new invention or a less expensive process, which may be the direct result of his efficient management, he has reduced materially the cost of his output.

In such an event, the resulting benefit should go to both the owner and to the rate-payer, and the owner should be recompensed for discarded property, rendered useless perhaps by the new invention, but, necessarily, put at once into the class of property no longer in use. It would be proper in such cases, without imposing hardship on the rate-payer, to allow the rates to remain as they would have been without taking account of the reduced cost of production, at least long enough to amortize completely so much of the original plant as the new invention has rendered useless. After this amortization is accomplished, the rates will be subject to such adjust-

ment as will bring the rate-payer into fair participation of the benefit which has resulted. If any policy less favorable to the owner was followed, he would find it to his advantage not to make any innovations which would require the abandonment of a part of his plant still in serviceable condition. He could not afford to assume new hazards resulting from the investment of more capital, unless he felt sure that his profits would be increased thereby. Mr. Grunsky.

JOSEPH MAYER,* M. AM. SOC. C. E. (by letter).—For a thorough investigation of valuation and the therefrom inseparable regulation of public utilities, the nature of the economic problem which must be solved and the kind of solution desired must first be defined clearly. Mr. Mayer.

In competitive private industry the owner, generally, is practically at liberty to choose the prices for which he is willing to sell his products, provided free competition is not in any way interfered with.

The public relies on competition to increase automatically the supply and thereby reduce the prices whenever they are such as to produce for a long time much larger average profits in any industry than are secured in other industries offering otherwise equal advantages.

Experience shows that, in many industries producing the necessaries of life, competition is extremely limited or impracticable, and that excessive prices, producing unusually large average profits on investment, thereby arise. Such industries are pre-eminently street railways, the distribution of gas, electricity, and water, telephone and telegraph service, the carrying off of sewage, local, and to a less extent long-distance, railways, and express service.

Capital is obtainable for these industries if the prices of their products or services are regulated so that average profits are the same as in competitive industries. The practical problem is to find such a regulation of monopoly prices as will equalize average monopoly and competitive profits without interfering with efficiency of management.

One method of price regulation is to introduce public ownership and operation, as is done universally with sewers, largely with water supply, and, to some extent, with others of the industries mentioned.

Public operation of monopolies, due to gross inefficiency of management, has often proved a failure. Private operation, with public control to secure fair prices without destroying efficiency, therefore, is attempted.

To secure efficiency with private operation, the net revenue of each enterprise must depend on its efficiency of management. This cannot be attained by regulating the prices so that the individual enterprises secure a uniform rate of return on the investment. For fairness, the prices must be regulated so that the average rate of net return on the investment in every regulated industry, not that in

* Hollywood, Los Angeles, Cal.

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individual enterprises, is the same as this average net return in such competitive industries at the same time and place as offer otherwise the same advantages.

The Committee advocates a standard rate of return on the investment in every individual normal enterprise, practically ignoring differences of efficiency. It defines a normal property or enterprise as one which is neither overbuilt, inadequate, nor improperly located. The Committee, though advocating and assuming in its ascertainment of the tangible value a regulation ignoring the existence of differences of efficiency and the consequent fairness of unequal rates of return, endeavors, in a hesitating manner, to correct the therefrom resulting errors in the chapter on "Intangible Values."

The aims pursued by the Committee and the writer become thereby much less divergent. The writer, however, claims that these aims cannot be attained by the circuitous method adopted by the Committee. The Committee tries to follow closely the decisions of the Courts in determining values. The writer believes that, as value is an economic phenomenon, it must be determined by studying economic laws, and not legal decisions merely. Economists, not Courts, are competent valuators. The Courts acknowledge this by only stating correctly what factors influence value and must, therefore, be considered, but refraining from prescribing how value is to be determined from these factors.

The Courts insist on fairness, and keeping promises is the most important guidance in determining what is fair. The writer, therefore, believes that the first task in valuation is to ascertain what promises have been given. This is a political and partly a legal question, wherein the laws and the decisions of the Courts are final. The second task is to find what values result from keeping promises, and this is a purely economic question. Court decisions which contradict economic laws cannot stand, and will be reversed as soon as this is clearly demonstrated.

The decisions quoted in part by the Committee on page 1372 indicate that the Courts sustain the determination of value in the ordinary economic manner, that is, from estimated future net revenue and market quotations, and do not sustain the theory that a public service property is entitled to a standard return on either actual cost or cost of reproduction less depreciation, which underlies the assumption that costs are values.

In competitive industry it is held to be fair that the investor obtains a greatly varying rate of return on his investment, the amount of which depends mainly on the industrial capacity of the promoters and managers of the enterprise, but partly on changes in the industrial world occurring independently of the management. Competition

is relied on to assure the fairness of prices. The independence of private ownership is only practicable when the owner secures the profit and loss resulting from judicious or injudicious choice of enterprise, from good or bad design, or from other differences in efficiency of management. The Committee proposes in its main argument that the fairness of regulation be judged by the return secured by individual investors. The writer maintains that, with private property, this is not practicable. The return of the individual enterprise must vary greatly with efficiency, and the efficiency can only be practically judged from the rate of return secured under conditions prescribed in some other way than by trying to secure a uniform rate of return in individual enterprises. Mr. Mayer.

The Committee not only advocates such a regulation, for natural practically complete monopolies, as will give a uniform rate of return in individual enterprises, but assumes in much of its reasoning that such, if any, regulation has actually existed in the past and ought to have existed where all regulation was absent. As a matter of fact, no such regulation ever did exist, and never ought to exist. Many of the Committee's inferences in regard to values, depreciation, and the proper future rates based on such assumptions, therefore, are in error. The activity of regulating commissions is generally limited to reducing grossly excessive prices where consumers complain. They then endeavor to fix such prices as will, in their opinion, with average efficiency of management, produce about the same rate of profit as in local competitive industries at the same time. The price adopted is mostly and necessarily, since the required facts are missing, an approximate guess, unsupported by adequate evidence. Actual values of enterprises thus regulated, therefore, are widely different from those which would have resulted from the imaginary regulation advocated by the Committee, and which it endeavors to determine.

Where competition remains an important element, no attempt is ever made to secure a uniform return on the investment in individual enterprises, for the very good reason that it would not be practicable. The fundamental principle which governs commerce and should govern regulation and valuation is: "Be careful what you promise and keep your promises."

In granting franchises, in the past, the public, generally, has not been careful in its promises, and has often made it very difficult or impossible, without gross injustice, to keep the promises it made. The economic developments of the present were not foreseen in the past, and the promises made were so extremely indefinite that it is generally impossible to determine accurately what was promised and to keep accurately the promises. In commerce, it is sometimes found

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just, under exceptional unforeseen circumstances, by a moratorium to relieve merchants of given promises to pay their debts when due.

The case of unreasonable public franchises is in some respects similar, and there are cases when justice requires expropriation on fair terms. Regulation was introduced that was not provided for in many of the franchises, and was frequently modified in such an unexpected manner as to affect seriously the net earnings and thereby the values of enterprises. Though the principle of keeping promises is thus frequently disregarded, it nevertheless remains a valid principle whenever it is possible to follow it without serious injustice. Another necessary principle governing valuations follows from the nature of value.

From the definition of value, cited by the Committee in the Glossary, it follows that the value of a public utility, the securities of which are largely dealt in on the exchanges, can be most closely obtained from a study of the quotations in the stock exchanges, and the amount of securities outstanding. The security quotations represent an estimate by the market of the present value of the future income expected to be paid to the owner of the securities.

This future income and the current quotations measure the value of securities, and they are what the owner values and what the Courts endeavor to protect. The future income depends largely on the future regulation. This latter is not exactly known, but its nature is inferred from the present legally established regulation and the knowledge that the Courts, generally, will not sustain such changes of the existing regulation as will destroy present values, as well as from the necessity of regulating prices so that it will be possible to secure the needed capital for building new properties, for extending old ones, and for rendering satisfactory service.

To ascertain the present values of future revenue, the rates of interest to be adopted must be known. Such rates must be chosen as will make, for enterprises the securities of which are largely dealt in on the exchanges, the value inferred from future net revenue the same as that inferred from the security quotations. The Committee proposes to find the values of public utilities by determining first the tangible and then also the intangible value, giving the latter due weight.

By strict logic, the total value is equal to the tangible plus the intangible value, and as the intangible value is equal to the exchange or market value, derived from estimated future net revenue and quotations, less the tangible value, their sum, or the total value, is equal to the exchange value.

The determination of the tangible value, therefore, is at best only indirectly useful for the purpose of valuations. The Committee pre-

fers, however, not to be logical. It infers, from the fact that the Courts give round sums for the intangible value, that the entirely indefinite phrase giving due weight to the intangible value must be substituted for the exact request to add the intangible to the tangible value. There is abundant reason for using round figures for the result of any valuation, however determined. It is improper to use exact figures when the degree of accuracy obtainable by any kind of valuation does not justify anything else but round ones. Another reason given is that the Courts do not assert that the value must be found by considering income and quotations alone. It might also be stated that the Courts expressly assert that costs are one element to be considered. The Courts, however, do not state how the value is to be determined from the various elements which must be considered. The cost and the average life of every perishable part of an industrial enterprise must be known in order to determine its annual decretion which is a part of the operating expense. The net earnings, therefore, cannot be determined without knowing the costs of the perishable parts of the plant. For determining what prices must be allowed to secure capital for enterprises capable of rendering the same service, and thereby to choose a fair regulation, the costs of a substitute enterprise are of importance. In a valuation based on net revenue, costs, therefore, are considered in two ways: First, in choosing the regulation to be assumed; and second, in determining the net earnings resulting therefrom. Mr. Mayer.

The Committee endeavors to show that the kind of regulation it advocates does not necessarily destroy efficiency. For this purpose it again departs from logic. It desires a regulation which makes tangible value equal to exchange value. This can only be attained by making the return on the investment proportional to its amount. It then recommends a departure from this uniformity by advocating the London and Massachusetts sliding scale of dividends.

The instance cited is the allowed 7% dividend for 90-cent gas with ½% increase of dividend for every cent of reduction in the price of gas. The gas company, under this rule, now obtains 9% dividends with 80-cent gas. In competitive industry, when a producer reduces costs of production he obtains himself the profit from the reduction of cost until the improvement is adopted by enough of the competitors to increase the supply materially; then prices and the profits of all producers are reduced, and the backward ones are compelled to improve, or receive abnormally low profits. After a short time, average profits are the same as before, and the public gets the benefit of the improvement. During the transition period, those in advance gained, those in the rear lost.

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With the sliding scale, when an improvement reducing costs is made by any one and widely adopted, all, whether fast or slow, obtain a permanent increase of dividends.

The inducement to make the first trial of improvements is greatly reduced; that to wait until they are well tried out is increased.

Improvements are discouraged, and permanent excessive dividends result from those adopted. The sliding scale is unfair to the public; therefore, it does not spread. The two requirements, fair prices and high efficiency, are not, and cannot be, secured by the methods of regulation advocated by the Committee. They can be secured by improved franchises for all natural monopolies.

The franchises should all be definite competitive operating contracts for limited periods of time.

Such a contract should be based on unit prices for the work performed, which work should be as accurately described and enforced as that described in a contract for building a large engineering structure. The contract must contain such clauses, referring to decretion and the compensation for the plant of the contractor, if any, at the end of the operating period, as will assure adequate maintenance and exact determination of the payments for such plant. If old, the property forming the subject of the operating contract must first be obtained by the public by a valuation based on the present value of the future net revenue with the present regulation, unless good reasons exist for believing that the present regulation is unfair. In this latter case, such a regulation, in harmony with promises made, should be assumed as will make it possible to secure capital for new properties capable of rendering equal service and for extending and satisfactorily operating old ones. If there is a good market for the securities, the value should also be determined from a study of the quotations.

For valuing the property and, later, for determining the annual decretion, an inventory must be made giving an estimate of the original cost, age, and average life and scrap value of every perishable part of the plant; or, where these are unobtainable, an estimate of the present value, the remaining life, and the scrap value.

For determining every year the decretion of the plant, which is a part of the operating expenses, the decretion must be defined so that it can be accurately determined and so that it agrees as closely as possible with the facts.

The following rules will serve for this purpose:

The nominal annual decretion of any part is the difference between its cost and its estimated scrap value divided by its estimated average life. The actual life and scrap value are different from the estimated average life and the estimated scrap value. To correct the errors

resulting therefrom, the following rules become necessary for calculating the actual decretion every year. Mr. Mayer.

For parts in use and younger than their estimated average life, take the nominal annual decretion; for parts in use and older than their estimated average life, take no decretion; for parts discarded during the year, take the cost less the decretion previously allowed for, and consider the actual scrap value as part of the earnings.

These rules take an estimated decretion for parts in use for which the actual one is unknown, and correct the therefrom resulting error as soon as the actual decretion becomes known by the discard and sale of parts. They, therefore, give as close an approximation as practicable of the actual decretion.

By means of the inventory and the rules for decretion received by the bidders, the decreted cost of the plant can be ascertained at the end of every year. If the decretion thus determined is considered a part of the operating expense and placed in a decretion fund to be used for replacements, improvements, and extensions, without thereby increasing the cost accounts, the plant will be efficiently maintained.

New capital is required for most plants, and any operating contract must contain clauses prescribing who is to furnish such capital for extensions and increase of capacity, and the compensation it is to receive. The public is interested in all parts of the plant which may survive the contract, and a board of engineers representing the public must have a controlling voice in selecting and supervising the acquisition of new plant. An operating contract must describe the kind of service wanted as well as the maximum prices to be charged, based, in an old plant, on the previous ones, with such changes as appear advisable at the time of framing the contract. The operating company should be permitted to reduce, but not to increase, the prices. The contract should be framed by representatives of the public. Invitations to bid should be sent out giving all the available information required by bidders to assist them in calculating the net earnings during the period of operation, which might be about 20 to 30 years. The company should be required to surrender the plant at the expiration of the contract at cost of the parts, if any, furnished by the contractor, with the approval of the representatives of the public, during the period of the operating contract, the decretion fund going with the plant.

The bidders should be asked to give the annual rental which they are willing to pay. The contractor must furnish a guaranty for at least one year's rent, which should be payable monthly out of the net earnings, if possible. If the guaranty fund is encroached on, the contractor should be required to replenish it. A quick foreclosure, in case of non-fulfillment of conditions, should be part of the contract.

Mr. Mayer. By such provisions, large contracts can be awarded safely to bidders backed by a moderate capital. This will make real competition possible for every large enterprise.

This, though quite incomplete, will give a general idea of the kind of definite competitive contract that should replace the present indefinite franchises, in which protection of the interests of the public is obtained by commissions having, among many other indefinite powers, that of prescribing and changing without compensation, after award of the franchise, the prices of the products and services furnished by the operating or owning company. Changes are necessary in any long-term contract. Therefore, there must be provision for making them without changing the value of the contract.

The two contracting parties should be on an equal footing, and their respective rights and duties should be defined exactly.

With such a contract, fair prices would be assured by competitive bids for contracts of such size as will assure active competition.

Efficiency is secured because the profit or loss of the contractor depends entirely on his efficiency. The drawing up and enforcing of such a contract does not require the superhuman knowledge and ability that would be necessary for assuring efficiency and fair prices by the present methods of regulation. Only one valuation is required for each old enterprise, that for acquiring ownership of the property. All succeeding transfers would take place for sums based on costs and decretions, defined in advance so that they can be determined accurately.

Reviewing the foregoing discussion: the problem to be solved is to replace adequately the self-regulating competition which has disappeared either partly or altogether in some fields of productive industry. Where free competition prevails, fair prices and efficiency are secured automatically by increased supply and falling prices, in industries where average profits are excessive, and reduced supply and rising prices where they are abnormally low. The fairness attained is not equal profits for good and bad management but, in the long run, approximately uniform average profits for the same degree of industrial capacity in different industries. Efficiency is secured by profits depending mainly on it. The return on the investment in the individual enterprises of every industry varies widely with the economic skill shown in their promotion and management, and partly with unforseeable economic changes outside and independent of the management. The board of directors, consisting of, or representing, the largest stockholders, has generally no other common primary aim than the, in the long run, largest possible return on the investment. The value of the business may be a small fraction of the investment or may exceed more than ten times its amount. An industrial enter-

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prise is a tool for the production of a future net revenue; its value is the present value of its future net revenue; this, on an average, depends mainly on the cost of production of the business, but, in individual cases, mainly on the industrial capacity of the management. This deviation of the value of such an enterprise from its cost of production is what produces efficiency of promotion and management without public control, and makes private property of such enterprises practicable.

The directors will endeavor to secure, and will retain, the most efficient managers. Were uniform profits guaranteed, as is proposed for the ideal regulation of the Committee of completely monopolized industries, a crowd of incompetent promoters would encumber the field of new enterprises, and the directors would probably choose their sons, nephews, and friends for the leading positions, and would follow the same method in selecting the minor officers.

Since, with uniform profits, the public pays for good or bad management and carries the risks, it will gradually insist on full control of promotion and management, independent private property will disappear, and public operation and ownership will result almost automatically. Uniform profits for individual enterprises are incompatible with private operation, because, inevitably, they destroy efficiency of management. The only possible alternatives, therefore, are public operation, and private operation with profits in individual enterprises mainly depending on efficiency and with average profits in every monopolized industry, the same as the average profits of competitive industries at the same time and place.

The frequent inefficiency of public operation in democratic communities arises from the method of choosing the public equivalent of the board of directors, and the general manager and his assistants. They are either elected directly or are appointed by representatives elected for other qualities than their knowledge of the industrial enterprises under public management. The voters and their representatives generally know little about these industries.

The management is controlled frequently by the most powerful politicians of the day, and will remain in power through its control of votes. It is often dependent on the politicians in its appointments and discharges of employees and the choice of their salaries and wages, in the prices charged for services rendered to the public, and sometimes, also, in its purchases of materials and supplies as well as in the choice of improvements and extensions.

In attaining and keeping in power, the efficiency of management is a very small factor, and, therefore, is largely neglected.

A private board of directors in a company with a fixed rate of dividends would be worse, but a private board with a rate of dividends depending mostly on efficiency is far superior.

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There is, therefore, no reasonable doubt that, as long as the evils described prevail, private operation will secure a cheaper and better public service when the net earnings of the private company depend mainly on efficiency of promotion and management. One way of securing the fairness defined as equal average profits in each monopolized and in competitive industries, combined with the efficiency of management resulting from profits in individual enterprises depending mainly on efficiency, consists in replacing the kind of competition which has disappeared, because it is impracticable, by another kind which is practicable. This consists in acquiring public ownership of all partial and complete monopolies, and selling definite operating contracts for limited periods to the highest bidder. Two necessary features of such contracts, which have been given insufficient attention in past franchises, are an exact definition of the annual decretion and of the price to be paid to the contractor or owner, either when he fails to comply with the terms of the contract or at the end of the operating period.

Another feature of such contracts, not mentioned in the foregoing, is the possibility of prescribing exactly in the contract, without injustice to the contractor, the minimum wages and other working conditions approved by public opinion, and the manner in which they are to be changed without changing the value of the contract.

Near-by economic changes, occurring independently of the contractor and influencing the net earnings, can be foreseen largely by the bidders and allowed for in the bids. Distant changes, which cannot be foreseen, will influence future bids in the same manner. Approximate proportion of efficiency and net earnings will be secured thereby. In such contracts changes in the value of the lands occupied by the enterprises accrue mostly to the public to whom they belong. They are caused largely by events beyond the control of the contractor. Justice requires, therefore, that they should not affect his revenue. In these respects public ownership with the private operation herein described is more just than private property in competitive enterprises. Another advantage is the much greater stability in the value of all securities representing part of the income of either the property operated or of the operating contract. With publicity of accounts and exact determination of the annual net earnings, the value of the contract can be judged much more accurately than that of a partial or complete monopoly regulated in the present necessarily arbitrary manner.

Transfer of the property from one operating company to the next, based on costs, when agreed on by contract, is far superior and is more just than transfer based on values. Costs can be ascertained accurately when followed from the start, values never; costs make the contracts

definite, values indefinite. This change from the general practice of using values instead of exactly defined costs, in defining the terms of transfer, is what makes short-term operating contracts practicable. The other method of securing fair prices, together with efficiency (which are both essential), is to fix, by regulating commissions, from time to time, the kind and quantity of service to be furnished by partial or complete monopolies and the prices which they are permitted to charge. Mr. Mayer.

The prices are fair when they make the average profits in every monopolized industry the same as in competitive ones; they secure efficiency when the profits in every individual enterprise are in proportion to efficiency of management. To fix the prices, it is necessary, therefore, to ascertain, first, the average profit secured in competitive industries at different times and places. By profit of an enterprise is meant the percentage of net earnings on its value. The sum of the values of a large number of well-selected competitive enterprises and the sum of their net earnings must be ascertained. The latter multiplied by 100 and divided by the former gives the average competitive profit.

Though the value of individual competitive enterprises varies greatly from their cost of production, the sum of the values of a very large number, representative of all competitive enterprises, is approximately equal to the sum of their costs of production. This results from the tendency of capital to flow to the most profitable investments, which makes the average return of all kinds of otherwise equally attractive competitive investments the same. The value of the sum, but not the value of each, can be determined, therefore, from cost of production. The average profit of competitive enterprises can be determined, therefore, if the necessary facts are ascertained.

With any given regulation, the average profit in any monopolized industry can be determined in a similar manner from the sum of the net earnings and that of the costs of production of the enterprises. The average profits thus ascertained should be the same in competitive and in every monopolized industry. This assures fairness of the regulation as a whole. If the profits are found to be different, the regulation should be changed so as to make them alike.

The regulation in detail must prescribe different prices in different places with different costs of production. The differences in cost of production in different places with ordinary methods of production, therefore, must be ascertained.

Although it is theoretically possible to regulate fairly in this manner the prices of monopolized industries without destroying efficiency, the practical difficulties are too great for the exact application of this theory. The profit of competitive industries varies from time

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to time and from place to place, and that of the various monopolies should vary in the same way. The necessary facts are unknown and cannot be determined accurately, and the necessary calculations would be extremely complicated. An enormous staff of regulators would be required to apply the theory.

The various regulating commissions cannot use this method, because they have neither the needed facts nor the staffs to gather them. Therefore, to simplify the method, a tendency toward uniform profits, where the management is not grossly inefficient, is inevitable. Regulation is generally only undertaken where there is complaint of excessive prices, so that the actual regulation is generally extremely sporadic and intermittent, and it does not follow consistently any theory, because this is practically impossible. The decisions of the regulating commissions, therefore, cannot be foreseen, and often appear to be quite arbitrary.

A very serious consequence of the, at present, extremely indefinite relations between the natural monopolies and the public is the uncertainty regarding the prices which will be permitted by the regulating commissions and the consequent large fluctuations in the market values of their securities. These commissions are directly or indirectly selected by, and are responsible to, the public, one of the parties to the implied contract between it and the utility companies. The principal protection of these companies is the impossibility of securing capital for new properties, extensions of old ones, and adequate service, unless sufficient return is allowed on the existing investment. The uncertainty of the return and the fear that it may not be adequate prevent new investment, unless a large return is allowed to compensate for the apparent risk. The regulating commissions may fix such prices and consequent returns as will attract capital and make it thereby possible to supply adequate public service. Such rates of return may be thought to be excessive by the public, unless the inadequate service with lower ones is felt to be the greater evil. Inadequate service, therefore, may lead to higher rates of return than adequate service, a condition which is evidently not conducive to the best service. Long-continued, inadequate service by private companies will create a demand for a radical change. Unless a way out, which will be an improvement over present conditions, is made widely known and adopted, a successful agitation for public ownership and operation seems to be inevitable. With definite competitive contracts any practicable and desirable kind of service can be obtained by the public by paying the necessary competitive price. As most of the risks incident to arbitrary regulation disappear with definite contracts, investments in them are comparatively safe, and will attract capital at low rates of profit. Most of the capital required would be secured by Government bonds at very low rates of interest.

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The law ordering the valuation of interstate railways by a Federal commission was the result of a widespread belief that the capitalization of these roads was largely fictitious and far beyond their cost of production. It ordered, therefore, the determination of the actual costs of production to date and of reproduction less depreciation, so that they might be compared with the known capitalization, in order to ascertain the difference between them.

The Committee has done an enormous amount of valuable work to ascertain the principles for determining these costs and the depreciation. It has not determined values, however, or the correct principles of valuation. What it calls tangible value is not a value, but a cost of reproduction less depreciation. In the writer's opinion, the Committee has not altogether abandoned widespread erroneous views in regard to the nature of value, though it gives, under value and market value, correct definitions of value, of price, and of market price.

The identification of value and price is harmless, as long as the value of money remains substantially constant. The corrections necessary with changes in the values of money are a separate problem, not discussed by the Committee.

The determination of costs meets the demands of the Federal law prescribing it. The unsatisfactory chapter on "Intangible Values" attempts to meet the demands of the Courts which insist on the consideration of revenue and market values. With a slight correction, prescribing the addition of what the Committee calls tangible value, and of intangible value to obtain the total value, its valuation is correct. The demands made by the nature of economic phenomena, which prescribe the highest laws to which all others must accommodate themselves, however, are but imperfectly met by the investigation of the Committee. The value of an industrial enterprise of any kind is the present value of its future earnings. Security quotations, where available, give the fluctuating opinion of the market on the value of the securities. There are often no reliable quotations; the estimated future earnings with a fair regulation furnish then the means of determining values.

For determining what is a fair regulation, costs and depreciations must be ascertained. The Committee has given the principles which must be followed in determining these, and has thereby done part of the work necessary for a correct valuation, but it has not given the principles which must be followed in a real valuation based on revenue and quotations. The task which is now being performed by the Federal Commission for Valuation is, in reality, a cost determination and not a valuation, which latter will not be attempted. The Commission thereby conforms with the spirit and real purpose of the Federal law. Its work will be very useful in improving present methods of regulation of public utilities, but will be inadequate for valuing them.

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W. R. McCann,* Assoc. M. Am. Soc. C. E. (by letter).—It is readily to be conceded that the Committee has produced a very creditable and momentous work. The task of developing any sort of unanimity from the chaos which has heretofore existed in matters pertaining to the valuations of railway and other public utility properties is an estimable performance that is appreciated best by those who are daily engaged in valuation work. The Engineering Profession as a whole, and particularly those members thereof who have to do with appraisals of public utilities, owe a sincere debt of gratitude to the authors who, by exhaustive research and by many conferences, have succeeded in producing a report which purports to represent and reflect the Committee's unanimous, composite idea of valuation theories. Such a report will carry great weight indeed, in the future, whenever it is quoted and referred to as an authority; and its two hundred pages will be fingered many times by youthful engineers, apprenticing in appraisal and valuation work. The extended scope of the report, and the important utilization of its contents, should render engineers quite cautious in concurring indiscriminately with all the statements and theories therein, even though advanced unanimously by the Committee.

It is not to be expected that the Committee could produce a report which would be generally approved by the Society's membership. It is much more probable, judging from controversies which have arisen over the subject matter in the report, that every statement therein has its opponent. The question is, "Does the Committee's report on valuation of public utilities reasonably represent the status of the art to-day?" The writer believes that the question must be answered in the negative, particularly from the point of view of the regulatory bodies and the judiciary, which are charged by statute with establishing equities between utility companies and their consumers.

In a great many respects, the report is quite favorable to the positions heretofore taken by corporations. A careful study of it makes manifest the conclusion that hardly a scrap of a claim which will result in enhancing the value of utility property is neglected, unless it is obvious, after many a try-out before commissions and Courts, that the claim manifestly is to be disallowed. There is great danger in following the report implicitly; young appraisers and students, particularly, may be led astray, with the result that they may recommend (often advocate) absurd conclusions derived from adhering too strictly, without the exercise of mature judgment, to the Committee's leanings.

To point out all the places wherein there is much more to be said in behalf of the public's side than has been stated by the Committee would consume as much space as the report itself, and even more.

* Washington, D. C.

Such extensive purpose is not the writer's intent in this limited discussion; he will confine his remarks to one important subject, that of "Original Cost to Date"—a matter which has been given but five pages, as contrasted with about a hundred pages (nearly one-half of the entire report) given to the antithetical theory, the "Cost of Reproduction." Mr. McCann.

It is to be frankly conceded that much of the language of the Courts to date has been favorable to the reproduction theory of valuing utility property; and it is also opportune to remark that a considerable portion of the judicial language has been misquoted, misrepresented, and misconstrued by partisan advocates. It is not to be overlooked, moreover, that the strength of original cost is to be found in commission decisions; and this is especially significant in that those who devote their entire time to the solution of valuation, rate-making, capitalization, and other problems affecting public utilities have marked leanings toward the equities inherent in the original-cost method of valuation. In the writer's opinion, the day is not far distant when the Courts, on review of a record which contains a commission decision embracing a valuation based on a properly compiled original-cost appraisal, will comment much more favorably on the original-cost method than they have on divers reproduction theories.

Heretofore considerable discredit has been given the original-cost method, due to confusion which has existed as to what is meant thereby. The capital account carried on a utility's books has been taken to mean the original cost of the property; likewise, it is argued that the purchase-and-sale price in a trade consummated years after the original construction represents the original cost of the property; also, the original sum invested in the nucleus of a property has been assumed to be representative of the original cost. These are not correct definitions. A proper original-cost appraisal is as capable of precise definition as is any reproduction theory. Before proceeding further with this discussion, it is well to bring out what constitutes a sound original-cost appraisal.

The California Railroad Commission,* submits the following definition of the term "original cost":

"The term 'original cost' means the actual expenditures chargeable to capital account, in accordance with the Interstate Commerce Commission's or this Commission's classifications, in cash or its equivalent in terms of cash, by the public utility for its property in the State of California, as of the date of the valuation."

Mr. Hammond V. Hayes,† a consulting engineer of Boston, defines actual original cost, thus:

"The actual original cost is the sum of money which was expended by the undertaking for the property now in use for the benefit of the

* Annual Report for the fiscal period from July 1st, 1915, to June 30th, 1916, p. 92.

† "Public Utilities: Their Present Fair Value and Return," pp. 123-124.

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public. It is not what the original property cost but rather what the present property cost. The expression 'original cost' is liable to convey a false impression. What is required in a valuation is the 'actual cost' of the property now in use. The term 'original cost' has been used so generally in decisions of courts and commissions, however, that it cannot be now eliminated. * * *

"The actual original cost should not be considered to be the cost of the first unit of plant used in a particular place or for a particular purpose. Items of perishable property, which are no longer in use or useful, cannot be considered as a portion of the property to be included in a valuation for the purpose of determining the fair value for rates. Such items of property have passed out of existence and their cost should have been removed from the books of the company as a portion of the value of its assets. If the business of the undertaking had been conducted properly, reserves for renewals would have been made. These reserves are obtained from users as a portion of the charges paid for the service. Manifestly it is unfair to the users for the company to demand from them rates sufficiently high to create a fund for the replacement of obsolete items and then include the cost of such obsolete items in a new value upon which new rates should be based."

Mr. Robert H. Whitten,* of Brooklyn, likewise defines actual original cost:

"Strictly speaking, actual cost means cost of original construction plus cost of additions and betterments. It excludes all expenditures for renewals and replacements including supersession due to obsolescence or inadequacy. It includes only construction, additions and betterments that are a proper capital charge under approved accounting principles. This conception of actual cost, however, is one that has in the past been very imperfectly comprehended. Correct accounting principles are of comparatively recent acceptance and application. The references made by courts to actual cost or original cost plus improvements show that in most cases they have loosely interpreted the term to include many things that are not properly a part of the actual cost of the present property. In certain decisions it is apparently assumed that actual cost or original cost includes discount on securities issued, exorbitant profits to promoters, cost of replacing worn-out or superseded property, dividends paid out of capital, money sunk in unsuccessful experiments. That is, the term is considered as an equivalent to book value inflated by financial manipulation or loose accounting. Considered in this light, it is little wonder that 'original cost' has been discredited as a standard of valuation.

"Actual cost properly considered is the most natural and in many respects the fairest single basis for the determination of fair value for rate purposes. A fundamental principle of public service regulation is that as the public service corporation devotes its property to a public use it may consequently be required to render the service at reasonable rates of charge. Rates of charge to be reasonable may not be in excess of the fair value of the service and may not be higher than necessary to produce a fair return on the property devoted to

* "Valuation of Public Service Corporations," pp. 82–83.

a public use. The measure of the property devoted to a public use is undoubtedly, in the first instance, at least, the money that the company has actually and necessarily invested, *i. e.*, the actual cost." Mr. McCann

Mr. Halford Erickson, ex-Chairman of the Wisconsin Railroad Commission, in a paper entitled "Depreciation and Its Relation to Fair Value", read before The Utilities Bureau in November, 1915, comments similarly on the definition and application of an original-cost appraisal, as follows:*

"By original cost in this connection it seems to me should be understood the cost at which the existing property used by public utilities in rendering service was acquired. By cost of reproduction is meant the cost of reproducing the existing property under prevailing conditions. The original cost of the existing property should be shown by the books and records of the utilities provided these have been properly kept and are still in existence. When the books have not been so kept or are not available, the original cost as thus outlined may be determined very much in the same manner as that in which the cost of reproduction is found. * * * .

"When the original cost of the existing property is desired it can be computed upon the same inventory as that used in determining the cost of reproduction and upon prices which cover the period when the property involved was put into the plant. Such price lists may be had partly from the records of the plant and partly from other sources. In this way the original cost of the existing property can be had with even greater accuracy than the cost of reproduction."

The following statement of the Committee (page 1355) follows the identical line of reasoning, to wit:

"As original cost to date, with comparatively few exceptions, is not the book cost of the property but the cost of the existing items, it will be necessary as a rule to make a schedule of the various existing property items, in the same way that one would be made for determining the cost of reproduction; then reference would have to be made to the accounts, to ascertain the unit costs of the items."

The term "Original Cost" is used by the Engineering Department of the State Public Utilities Commission of Illinois to mean the actual expenditures, in cash (or its equivalent in terms of cash), made by a public utility for used and useful property which is properly chargeable to capital account and is embraced in an actual inventory; or, in the absence of records and books of accounts showing the actual expenditures, the estimated cost of the property as of the sundry dates of the installations of the various items of property is to be considered the original cost thereof.

Bearing in mind the aforesaid definitions of what constitutes a reasonable original-cost appraisal, particularly the definition set up in the preceding paragraph, it is well to refer to certain passages

* *The Utilities Magazine*, 1–3–113.

Mr. McCann. of the report, wherein the strict reproduction theory is modified and violated in principle, and recourse is taken in effect to the original-cost method. Perusal of the report, moreover, would seem to indicate that the principles of a pure reproduction theory are adhered to whenever it is to the general advantage of the property under consideration, and that historical reproduction cost (another name for an enhanced original cost) is resorted to whenever the strict reproduction theory fails to give the "top-notch" figure, or fails to arrive at the known original cost.

The first specific example wherein the Committee recommends a departure from the strict reproduction theory, in favor of the historically modified reproduction theory, is found on pages 1363 to 1765 (inclusive) of the report. Here there are cited instances of three different dams and reservoirs which were built under entirely different circumstances respecting contingent and auxiliary costs. The Wachusett Reservoir required the obliteration of several buildings, highways, railroads, and other property formerly on the present reservoir site. The Kensico Dam, built within the limits of an existing reservoir, required the construction of two temporary reservoirs to maintain a water supply. The Au Sable Dams were constructed under simple circumstances, unaccompanied by exceptional costs of damages, removals, or temporary works. The Committee asks if it is fair, for the reasons stated, "to assume like conditions to govern the estimate of reproduction cost to be used as a basis of 'fair value' ".

It certainly is apparent that these three dams and reservoirs are not to be appraised under "like conditions." The strict reproduction theory, however, calls for an appraisal of the existing property as it is to-day. Such a premise is satisfactory, say, for the Au Sable Reservoir, which did not encounter any unusual auxiliary costs during its construction. The strict reproduction theory, however, will not produce sufficient value for the Kensico and the Wachusett Dams; therefore the Committee, following in the footsteps of experts who have forsaken and modified the strict reproduction theory, "is of the opinion that all such items of cost, due to damages, destruction of property, purchase of rights, and temporary works, as cited in these illustrations, are clearly proper items to be included in the reproduction estimate when capable of historic proof", and this despite the fact that original-cost appraisals of these dams would reveal, in a very few items, the exact total sums expended in each of the three works.

It is not amiss at this point to contrast the proposed historical reproduction method, advocated by the Committee, with the original-cost method defined hereinbefore. In the Wachusett Dam, for instance, the appraiser must make a complete survey of the dam and reservoir as it is to-day; then he must apply unit costs of labor and material

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at prevailing prices to-day. Thus, the appraiser derives the reproduction estimate of the existing structure, whereupon he starts to conjecture what may have happened during the years of construction. Imagination is brought into full play in picturing the day-by-day and month-by-month changes in the scenery before him—once an industrious valley, teeming with activity, possessing thriving railroads, and incrusted with dust from many roads, now simply a peaceful lake. Many things might have happened during this transformation, and therefore the appraiser is restrained in his imaginative process to what actually did happen. The records of the work must be consulted, inasmuch as fascinating conjectures must be proved before commissions and Courts. Should the appraiser wander very far from the recorded facts and the actual cost thereof, he is in difficulties, if a keen counsel cross-examines him thoroughly as to the details underlying his theory. How much simpler it is to set forth the book-recorded costs and quantities of a few principal items, such as clearing, sanitation, excavation, temporary works, construction camp, masonry, earth fill, spillway, moving structures, moving highways, moving railways, etc. In arriving at fair values, commissions and Courts are not in the habit of disallowing sums which have been prudently and wisely expended in the construction of used and useful utility property; instead, their problem is to isolate the reasonable from the limitless conjectures of experts and from the flowery arguments of attorneys who seek to inflate the much-abused term "value."

The remarks of the last paragraph apply equally well to several other specific examples set forth by the Committee. The instances of the Pennsylvania and New York Central Terminals, on page 1365, are very similar to the aforesaid dam examples. There is little doubt in the writer's mind that excellent cost data are available for the recently-constructed New York terminals. Why is it necessary to disregard the actual original cost of these terminals and turn to a conjectural reproduction thereof? Opponents to the reproduction theories will answer that the underlying reason lies in the possibility of claiming inequitable increments in the railway's capital account. Advocates of the reproduction theories will answer that the purpose is to arrive at present value; and then the deluge of arguments of what constitutes value soon creates mire sufficient to obscure the original issue.

The next example cited by the Committee is that of a certain highway crossing a railroad track. For the sake of brevity, the Committee's statement (page 1366) relating thereto is reproduced, as follows:

"One illustration brought to the attention of the Committee is a case of a highway which originally crossed a steam railroad at a very acute angle, making a dangerous crossing to both railroad and

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public. At its own expense the railroad acquired land parallel with its own, changed the course of the road for several hundred feet and made a right-angled crossing. It was argued that, 'in case of reproduction, all other property remaining as at present', the highway would be in its new location and would not have to be reproduced. On this basis, many items which the owner of a railroad or other public utility was compelled to pay for might be cut out of an appraisal. But the highway would be in its original place were it not for the act of the railroad, and would have to be moved by any railroad, building presently, if the existing road had not been built; the evidence of the necessity, however, having been destroyed in the doing. The cost was a proper cost, capable of historic proof; and the Committee believes that it should be included as a cost in the estimate of reproduction."

If the writer understands this last illustration correctly, the cost of the old highway crossing (which had grown inadequate to meet modern traffic condition, and was dangerous), should be charged against the railway's accrued depreciation fund, created for the purpose of replacing equipment, structures, and other property as it wears out, grows obsolescent, and becomes inadequate. The actual cost, borne by the railway, of the new and re-aligned highway crossing should be placed in the capital account of the carrier; and such an item would appear in simple terms in an original-cost appraisal, without theorization of what may have happened during a process of reproduction.

It seems needless to dwell further on the several instances wherein the Committee advises departure from the strict reproduction theory and recommends the historical modification thereof. The writer regrets that the Committee did not see fit, in each of the examples, to point out wherein the original-cost method would establish more convincing equities before a tribunal in a valuation proceedings.

Before leaving the Committee's specific examples of application of a reproduction theory, attention is called to the stone bridge piers, instanced on page 1373 of the report, and to similar examples following. Certain abutments of a railroad bridge, built of stone masonry at a cost-to-reproduce of from $12 to $16 per cu. yd., in accordance with the road's practice for the previous 10 years, would now be built with concrete at a cost of from $7 to $11 per cu. yd. The Committee's conclusion, as to this and similar examples, is that the precise existing structure should be appraised at its reproduction cost, and that the only departures (page 1374) from such a rule are the cases where, due to obsolescence of existing machinery, it is absolutely absurd to pursue this method to its ultimate conclusion. Once again comment is forthcoming to the effect that the Committee's recommendations result in an adherence to the strict reproduction theory whenever the value to be derived therefrom enhances the original cost of an item of property; but, if the strict theory does not establish sufficiently high values, it would seem that the Committee's idea is to modify the theory. If a

theory can be modified in one place, there should be no great objection to a reasonable modification at another place. Unfortunately, multitudinous modifications of the strict theory result in individual theories of each and every appraiser. The writer, in the instance of the masonry abutments referred to, would call attention to the equities of appraisal at the original cost of the structure, regardless of what similar stone masonry is costing to-day, and regardless of the present-day cost of better concrete abutments. Mr. McCann.

Possibly it will not be amiss to cite here a pertinent criticism made by the late John M. Eshleman, formerly Lieutenant Governor of California, and once President of the California Railroad Commission, in an address* delivered before a conference held under the auspices of The Utilities Bureau, at Philadelphia, in November, 1915:

"* * *. The inconsistency of adopting the historical method with reference to some elements and rejecting it with reference to some others need only be mentioned. Why it is that the committee of presidents of the railroads of the United States should concern themselves with hidden costs of some elements of the properties of the railroads and forget all about hidden or other costs of lands and properties that have appreciated, is beyond my comprehension.

"I really seriously wonder at the logic of those who urge that cost shall ever have anything to do with this question unless they admit it has everything to do with it, and I marvel at their regard for the simplicity of public authority when they urge that original cost shall always be used even in their reproduction theory when it will give them more than present cost to reproduce, and at the same time utterly repudiate costs of lands and similar properties when such costs are less than the market value now obtained. I do not for a moment contend that the work of the engineers in making inventories, and, if you please, in fixing unit prices is not very important. But holding, as I do, that where government is dealing with monopoly it of necessity must deprive such monopoly of the power of taking all that it can get from its patrons. I am driven to conclude that government must determine as a substitute what the agency ought to take, and in determining what the agency ought to take, the safest and most just guide is what the agency has sacrificed and what service it has performed for the public. * * *"

That the original-cost method possesses merit in valuation proceedings is shown by such facts as (1) a more or less general adoption by regulatory State commissions of original-cost as a perfectly legitimate line of inquiry to shed light on property values, (2) the recognition by Congress of original cost as one of the methods to be pursued in the valuation of the nation's railroads, (3) the National Association of Railway Commissioners' unanimous insistence that original cost be considered in the Interstate Commerce Commission's valuation of common carriers' properties, and (4) Court decisions supporting the posi-

* *The Utilities Magazine*, 1-10-11.

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tion of the Supreme Court of the United States in Smyth *v.* Ames (cited by the Committee). No judicial authority, moreover, is to be cited to show that an estimate of the cost of reproducing the property (with or without deduction for accrued depreciation) is the sole and only guide to a reasonable and adequate valuation of a utility property for rate-making purposes. On the contrary, the inconsistencies of the reproduction method have been scored time and again. It is only recently that the Supreme Court of the United States, in the Des Moines Gas Case, repudiated the reproduction method when applied to what, in valuation work, is commonly termed "undisturbed paving."

In addition to those steadfast older commissions and their representatives, who have given due and just weight to the actual original-cost methods, it is noteworthy that many a recent author is outspoken in recommending the sacrifice of the investor as the measure of fair value, particularly fair value as a rate base. (Cf. Whitten and Hayes, *supra.*) A recent engineering contribution is "Public Utility Rates," by Mr. Harry Barker, Associate Editor of *Engineering News,* wherein the author (at pages 37 and 38) states:

"Present tendency seems to be to give more and more weight to figures of investment and investors' sacrifice in determining rate-basis worth. With the general imposition of proper accounting systems figures of investments and sacrifice can be more completely secured than in the past so that a better basis for value by investment can be established. Where regulation has been longest established, there investment has greatest weight; it is not unreasonable to expect that in the course of time the entire country may accept the theory. * * *"

The most common argument against the use of the original-cost method of valuing public utility properties, aside from the difficulties alleged to be encountered in securing records of such costs, is that appreciations in value of property (if such exist) are denied to the owners thereof. The reproductionists, however, do not point to any rule of common equity which entitles them first to earn an adequate return on their investments and then to participate in the profits which accrue to appreciation; but they rely on certain judicial passages, which in themselves are sound, although subject to linguistic abuses when applied unadvisedly to valuations that are made bases for rate schedules. Under certain rulings of the Courts, it may be argued that, even though a utility may steal equipment without being apprehended, and may convert that equipment into used and useful property in the service of the public, the stolen equipment must receive due recognition in a valuation and rate-making proceeding. Whether or not this view will prevail ultimately, under a continuance of State regulation, is a somewhat debatable question at the present time. A simple case will serve to illustrate the fallacy of too great weight given indiscriminately to appreciation in utility property. Assume that an electric plant, in a

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State where the laws provide for State regulation, costs $100 000, and assume, further, that the regulatory body of jurisdiction, after investigation at the very start, has fixed rates such as will yield full operating expenses plus 5% per annum ($5 000) for accruing depreciation and 7% per annum ($7 000) for a fair rate of return. At the end of 5 years, provided no change is made in the electric property, the utility will have accumulated $25 000 (plus earnings) in a depreciation fund, and each year will have paid a full and adequate rate of return on the entire investment. During these 5 years, if perchance the prices of labor and materials advance so that the estimated reproduction-cost-new of the identical property at the end of the period, according to expert appraisers, would be $110 000 (exclusive of betterments and additions), there will have been an unearned increment, over and above a fair rate of return, amounting to $10 000 in the value of the property—equivalent to $2 000 per year, or 2% annually on the original cost. In other words, a valuation thus made at the end of the said 5 years, resulting in revised rates being fixed on an estimated reproduction theory, capitalizes an unearned increment which automatically results in rendering a 9% rate of return throughout the entire first 5-year period. What has the utility done or denied to itself in order to deserve this unearned increment? Is not the public entitled to participate in the appreciation of property, at least to the extent of not having the same capitalized against it, to be borne by the rate-payers of the future? Carrying the illustration still further, let it be assumed that the estimated reproduction value sinks to $90 000 at the end of 5 years; then the reverse is true, and the utility each year is deprived of just earnings equivalent to 2% of the cost of the property. Under such conditions, would it not be argued that this deprivation of earnings would constitute a confiscation of property?

Such illustrations are indicative of the public's vital interest in what is termed "appreciation." It is to be borne in mind that "depreciation", as applied in valuation work, in no manner is the opposite of "appreciation." An original-cost valuation, if properly compiled, does not presume to inflict on a utility the losses occasioned by decreases in prices of material and labor. An original-cost valuation, however, does presume to reflect the conditions under which the bargain between a utility and its consumers was consummated. Under an original-cost valuation, the public sustains all losses due to the falling off of prices, although it participates in gains only to the extent of not having an unearned increment capitalized against it.

The reproduction theory has its place. It is undoubtedly one evidence of value, and an appraisal predicated on the strict reproduction theory should be made a part of every rate case; but the reproduction theory (or some modification thereof), under no circumstances, should be construed as the sole and only guide to value. Ultimate value for

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rate-making purposes is a discretionary exercise of judgment on the part of those governmental agencies which are empowered by legislative enactment to regulate public utilities, subject of course to judicial review should the administrative power be abused.

The Courts have not ruled that reproduction-cost-new (or less depreciation) must be the criterion by which to judge the present value of utility property for rate-making purposes; neither have the Courts stated that such reproduction-cost-new (or less depreciation) is not to be used, nor that original cost is not to be used. What the Courts have said is that all these evidences of value are elements to be considered with other factors in fixing on value for rate-making purposes. Whenever reproduction advocates turn their attention to bringing forth the advantages of the original-cost method as earnestly as they have expounded the reproduction theory, the disadvantages of the reproduction theory, which make it necessary to resort to original conditions and to original-cost conceptions, will disappear miraculously in a strict adherence to the actual original cost method of valuation. A fair rate of return on the actual dollars prudently expended for utility property that is used in and useful to the rendering of public service are words that jingle with equity; and it is the writer's opinion that judicial sanction, supporting the position of commissions respecting original cost, is not long to be withheld.

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Paper No. 1402

THE DISTRIBUTION OF STRESSES IN MITERING LOCK-GATES, WITH SPECIAL REFERENCE TO THE GATES ON THE PANAMA CANAL

BY HENRY GOLDMARK, M. AM. SOC. C. E.

SYNOPSIS.

This paper is based on an extended study of the laws which govern the distribution of stresses in different parts of a mitering lock-gate under service conditions. For reasons mentioned in the paper, the problem is indeterminate, so that the solution was based on the theory of elastic work.

A general method of computation is first outlined, which is simple in principle, though the computations are somewhat laborious. It is believed to be more complete and accurate than any previously developed.

An application to several of the lock-gates on the Panama Canal follows. The resultant loads which are sustained by the different horizontal girders and by the vertical bracing are computed in detail, as well as the pressures exerted by the gates against the bottom sills.

Some practical conclusions, drawn from the results obtained, are also given, as a guide for future designs.

General.

Mitering gates have been used in canal locks for several centuries, and are still the preferred form, to the virtual exclusion of other types. In large modern locks, they are generally built of steel, and are among the most expensive structures used in hydraulic works. In the interest of economy as well as safety, therefore, careful designing, based on a correct determination of the stresses, is a matter of importance.

The problem of finding the stresses in such gates is intricate, and has been the subject of much investigation in the past. In connection with the design of the gates for the Panama Canal, the largest yet built, the writer had occasion to make a rather detailed study of the subject. It seems proper to put on record some of the results obtained, especially as American literature on lock-gates is very scanty. It is believed that the method of calculation used is novel, and is an advance on previous practice.

The stresses are of two kinds, those due to the weight of the gate leaf, and those due to water pressure. The former do not affect the design of the gate seriously, except in the parts which serve to support it on the foundation, and to connect it to the anchorage in the walls. They are greatest when the lock chambers are emptied to permit of inspection and repairs. Their computation is quite simple when the weight of the metal work is known.

The stresses due to hydrostatic pressure, on the other hand, govern the dimensions of the principal members, as well as most of the secondary parts. The external loads in this case, too, are entirely definite, as well as their points of application, but the stresses they produce are somewhat indeterminate. This is due, in large part, to the complex structure of the leaf, which has numerous horizontal and vertical members riveted together at their intersections and covered by a continuous sheathing. The primary function of the different members, in transferring the hydrostatic load to the masonry, is well defined, but the rigid connections cause the deflection and, hence, the stress in any

one member to depend, not only on its direct load, but also on other parts of the gate.

The distribution of the stress, therefore, is complicated, even in single-leaf gates which span the lock at right angles and support the water pressure by beam action.

In miter-gates, which act as arches, there are further difficulties. The first arises from variations in the surface of contact on the miter and quoin-posts, where the gates bear against each other and against the hollow quoin. Especially if timber bearing pieces are used, the exact point at which the reaction acts, is uncertain, so that it is necessary, in the computations, to assume it to have a considerable deviation from the center of the bearing, in order to obtain the maximum stresses.

A further difficulty arises from uncertainty as to the pressure exerted by the gate against its sill. In single-leaf gates this will not change appreciably after the first adjustment is made. In miter-gates, however, there are wide variations in the sill reaction at different times, with corresponding changes in the stresses. This is due to changes in the length of the leaves arising from variations in temperature, wear in the contact pieces on the gate posts and sills, and from various minor causes. Even with careful workmanship and fitting, it is hardly possible to ascertain the sill pressure exactly, although, as will be shown later, it is feasible to arrive at limiting values, not likely to be exceeded in practice.

In this paper, a method of calculation is developed, by which the sill reaction and the loads on the different horizontal and vertical girders may be found in a given gate for an assumed hydrostatic loading and various conditions of sill contact. After these loads have been fixed, the stresses in the different members can be computed without special trouble. The gates are assumed to be of steel and "horizontally framed" with numerous horizontal girders. In the gate with "vertical framing", which has only two main horizontals, the stresses are more determinate, and the theory is much simpler.

The problem is first stated in general terms, and a solution is found applicable to gates of varying dimensions, girder spacing, and cross-sections. It is then applied to the largest and the smallest of the

Panama gates. Finally, some practical conclusions derived from the computations are given, as a guide in future designs.

Statement of Problem.—A mitering lock-gate consists of two leaves, which together support the water pressure due to the difference of head on the opposite sides of the gate.

If there is no contact at the bottom sill when the gate is closed and under pressure, the entire hydrostatic load is transferred by arch action to the side-walls. On the other hand, if the leaves bear against the sill, the latter will carry a part of the load, the remainder being, as before, supported by the lock walls.

The proportion of the total load which will go to the sill, in any given case, depends on the structural arrangement of the gate frame and the relative adjustment of the gate and sill.

The object of the investigation given herewith was to determine, for the Panama lock-gates, the pressure of the gates against the sills for different adjustments, the distribution of the loading between the several horizontal girders, and the stresses in the vertical framing.

Previous Investigations.—The determination of the laws which govern the distribution of loading in the horizontal and vertical members of mitering-gates has been made the subject of extended studies by several distinguished engineers in the past.

M. Chevallier* made what was perhaps the first study of this subject. It included a series of tests with wooden models. The conclusions he drew from his experiments, as to the interaction of horizontal and vertical members in gate leaves, are in entire accordance with subsequent investigations, and of much interest even now. He gave no general formulas or rules applicable to the large gates of modern times.

In 1867, M. Lavoinne† published a mathematical investigation covering the same subject. His method is very complicated, although applicable only to gates having equal horizontals spaced at equal vertical distances. Owing to these assumptions, and for other reasons, Lavoinne's formulas are not applicable to large modern gates, in which the cross-section and spacing vary from the top to the bottom of the leaf.

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* *Annales des Ponts et Chaussées* for 1850.
† *Annales des Ponts et Chaussées.*

In 1887, M. Galliot* presented a new mathematical study, making practically the same assumptions as Lavoinne, but attaining somewhat simpler results.

In his treatise on "Mitering Lock Gates", published in 1892, First Lieut. (now Brig.-Gen.) H. F. Hodges, U. S. A., gave a discussion on vertical framing on a somewhat different basis, and deduced valuable rules for its practical dimensioning.

Method of Calculation Used in the Present Investigation.—The method herewith presented was developed by the writer in a less perfect form, in 1899, in a study on lock-gates made for the Board of Engineers on Deep Waterways.

The solution is based on the well-known "method of least work." It takes account of irregular spacing in the horizontal girders, as well as of variations in their cross-sections, and considers the cross-bending as well as the direct compression in the girders. In case timber is used for cushions at the miter and quoin posts, or at the sill, the formulas obtained can easily be modified, so as to allow for the difference of material.

Although the method is correct in theory, the unavoidable lack of homogeneity in the steel, the difficulty of determining the vertical and horizontal rigidity of the leaf exactly, still more the uncertainty as to the relative adjustment of the gate leaves and the sill, prevent a very close determination of the actual stresses.

It is believed, however, that the results obtained are reliable within reasonable limits, and will prove of much use in analyzing the strength and stiffness of existing gates or proposed designs. It should be added that the formulas when applied to the gates of the Poe Lock, at Sault Ste. Marie, gave results agreeing quite closely with deflection measurements made by the writer.

Like most applications of the elastic theory to complex structures, the method of least work cannot precede, but must follow, the complete design. In other words, it is necessary to adopt a detailed arrangement of all parts and afterward determine the distribution of the stresses in the different members.

Effect of Vertical Stiffness.—A gate leaf consisting only of a certain number of horizontal girders and an absolutely flexible sheathing would have no vertical stiffness. Such a gate would transfer no load

* *Annales des Ponts et Chaussées.*

to the sill, except the water pressure which acts on the lower half of the bottom panel and is carried directly to the sill by the sheathing. Each horizontal girder would support simply the load which corresponds to the hydrostatic head due to its position.

In practice it is impossible and undesirable to build a gate without vertical rigidity. The sheathing, combined with the quoin and miter posts and the intermediate vertical brace frames and intercostals, forms a vertical girder of considerable strength. By its resistance to bending, this girder modifies the loads on the different horizontals, making them greater or less than those corresponding to the hydrostatic pressure. As a rule, the vertical girder transfers a part of the total water pressure to the sill.

For purposes of calculation, the leaf is taken as consisting of a horizontal and a vertical system of girders crossing each other at right angles. The horizontal system consists of the several main girders or arches, spaced as they are in actual construction. The vertical system is assumed to be equivalent to a single girder extending continuously over the whole length of the leaf from the quoin to the miter post. Its flanges are formed by the gate sheathing, and its web is equivalent in total cross-section to the web plates in the several vertical frames and end posts. This simplification is justified by the close spacing of the vertical frames and intercostals, which prevents the skin plates from buckling.

Sill Contact.—In case there is no contact at the sill, even when full water pressure acts, the entire load, of course, will be carried by the horizontal arches to the side-walls.

If, in such a gate, all the horizontals are proportioned to support the hydrostatic head with exactly the same unit stresses in the steel, they will all have exactly the same deformations under load. There will be no bending stress in the vertical girder, so that it will remain straight, even after the gate is supporting the water pressure. The loads on the several horizontals will be those corresponding to their hydrostatic head.

In practice, the horizontals near the top are always stronger than theory requires, in order to ensure increased safety against accidental blows and to avoid the use of unduly small rolled shapes. It is also hardly possible to design all the other horizontals so that they shall sustain exactly the same unit stresses. There will always be some

variation, therefore, in the deflections of the different horizontal girders. This will produce a tendency to bend the vertical girder which is rigidly connected to the horizontals, and its resistance to bending, in turn, will affect the deflections and modify the loads of the horizontal frames.

However, the effect of vertical rigidity, when there is no sill contact, will be very small, except at the extreme top of the leaf.

The case of "no contact" should always be provided for in the design, for, from various causes, it is likely to occur in all mitering lock-gates as either a temporary or permanent condition.

In ordinary cases there will always be a greater or less reaction at the sill. If the water on the up-stream side extends to the top of the gate, and there is no lower pool, the greatest sill pressure theoretically possible would be equal to two-thirds of the total load supported by the gate. This maximum can only occur when the adjustment is so inaccurate that, even with continuous contact along the sill, the two leaves will touch only at the very top of the miter posts, even when the gate is subjected to the full head of water. This is an extreme case which would seriously overstrain the gate, and can be avoided by ordinary care in adjusting the leaves and sill.

A much smaller reaction may be counted on as a practical maximum. It seems quite safe to assume what is sometimes called "perfect contact", that is, continuous contact along the quoin and miter posts and also along the sill when the gate is closed, but before it is subjected to water pressure.

With both timber and metallic bearings, the actual conditions will probably correspond to lower sill pressures, as there will rarely be absolute sill contact in the dry.

Therefore, two conditions of adjustment at the sill were considered in the computations:

(1) No contact at the sill, even with full head;
(2) Simultaneous contact at the sill, miter, and quoin posts, before the water pressure is applied.

Let Fig. 1 represent the vertical section and Fig. 2 the plan of a gate leaf, of length, L, consisting of $(n + 1)$ horizontal arches and a continuous vertical girder, the stiffness of which is assumed to be uniformly distributed over the length of the leaf. The sheathing is

supposed to carry the water pressure directly to the horizontals, and the connections to be such as to permit the transference of horizontal reactions between the arches and the vertical girder at their intersections. Let the magnitude of these reactions be denoted by X_0, X_1, ... X_n per linear horizontal unit of leaf. They will be both positive and negative in direction, and will act normally against the arches exactly as water pressure does.

If, further, P_0, P_1, ... P_n, are the direct water loads on the several arches per linear unit, their resultant total loads will be:

$$(P_0 + X_0)\ L\ (P_1 + X_1)\ L \ldots\ldots (P_n + X_n)\ L.$$

With no contact at the sill, all these loads are carried by arch action to the hollow quoins; but, if the lowest arch bears against an absolutely fixed sill, that arch will carry no load to the side-wall, and $(P_n + X_n)\ L$ will become the sill reaction.

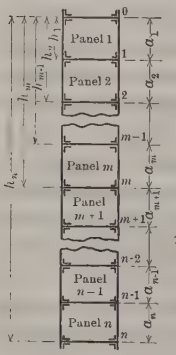


FIG. 1.

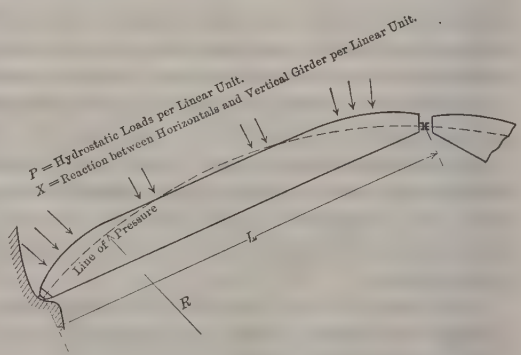

P = Hydrostatic Loads per Linear Unit.
X = Reaction between Horizontals and Vertical Girder per Linear Unit.
Line of Pressure
R

FIG. 2.

In any case the only forces acting on the vertical girder will be the transverse loads,

$$X_0\ L,\ X_1\ L, \ldots X_n\ L.$$

The static conditions of equilibrium as applied to this girder give only two equations:

$$\Sigma\ X = X_0 + X_1 + , \ldots\ X_{n-1} + X_0 = 0 \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

$$\Sigma\ M = X_0\ h_n + X_1\ (h_n - h_1) + \ldots X_{n-1}\ (h_n - h_{n-1}) = 0 \ldots . (2)$$

for determining the values of X_0 . . . X_n, although there are always at least three such unknown quantities.

Similarly, if the gate as a whole is considered, the reactions of the horizontals against the side-walls are indeterminate, as their number is in excess of the number of equations which can be obtained from static conditions.

The stresses in indeterminate structures of this kind, however, can be found by an application of the method of least work. The principle on which this depends may be briefly explained as follows:

If a perfectly elastic structure is subjected to external forces, the fibers in its parts will be deformed until a new condition of equilibrium is reached. The work done in this deformation (the internal work or elastic potential) will always be equal in amount to the external work, that is, the work done by the external forces.

By the principle of least work, the reactions and internal stresses corresponding to the new condition of equilibrium, in addition to being consistent with the statical equations, must be such as will make the total internal work done by the structure, in passing from its original to its new condition, a minimum.

This principle is sometimes derived from the theory of virtual displacements, but may almost be considered axiomatic, representing the theory of equilibrium in its most general form as applied to elastic solids.

The application of the method of least work to this problem consists in stating the work of internal deformation in terms of the known external loads and the indeterminate quantities X_0, X_1 . . . X_n, and finding those values of the latter, which, besides satisfying the static Equations (1) and (2), will give a minimum value for the total internal work done, while the gate passes from a condition of no stress to that in which the full water pressure is supported.

The internal work in the whole leaf will consist of the following parts:

(1) That due to arch action in the horizontal girders, involving generally both direct compression and cross-bending; and
(2) The work of bending the vertical girder.

The work done as the result of the shearing stresses in the arches and the stresses in the web members of the vertical girder, being relatively quite small, may be neglected.

The bending in the horizontal girders is due to the eccentricity of the line of pressures or resultants with reference to the center of gravity at the different cross-sections. For arches with continuous curvature, this eccentricity is quite small, but occurs to some extent in all gates.

If U_h = the internal work in the horizontal, and U_v that in the vertical girders, then,

$$U = U_h + U_v \text{ is the total internal work.}$$

For any elastic solid under purely axial stress (that is, direct compression or tension), the work of deformation will be:

$$U_s = \int_0^L \frac{T^2}{2\,E\,F}\,d\,l \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

and, for one subject to cross-bending only (beam action),

$$U_b = \int_0^L \frac{M^2}{2\,E\,I}\,d\,l \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

in which equations:

L = the total length of the member;

F and I = the cross-sections and moments of inertia at any point;

E = the modulus of elasticity; and,

T and M = the total axial force and bending moment at any cross-section.

From Equations (3) and (4) a general expression for the work equation for all parts of a mitering lock-gate (such as is shown in Fig. 1) may be written:

$$U = \sum_{m=0}^{m=n} \left[\int_0^L \frac{T_m^{\,2}}{2E\,F_m}\,d\,l + \int_0^L \frac{M_{hm}^{\,2}}{2\,E\,I_{hm}}\,d\,l \right] + \sum_{m=1}^{m=n} \int_0^{a_m} \frac{M_{vm}^{\,2}}{2\,E\,I_{vm}}\,d\,l \quad \ldots (5)$$

In this equation, m corresponds to any horizontal arch and also to the panel of the vertical girder just above the arch so denoted. The axial thrust and bending moment at any point in a horizontal are represented by T_m and M_{hm}, respectively; and M_{vm} is the bending moment at any point in the vertical girder. The first two terms give U_h (work in all horizontal arches), the integration being for the length of each individual arch and the summation to include the

$(n + 1)$ different arches. The last term gives U_v (work in vertical girder), the integration being for each separate panel, while the summation includes all of them.

The cross-sections, F_m, and the moments of inertia, I_{hm} are the average values for any given arch, and I_{vm} is taken as uniform for any given panel of the vertical girder.

As will be shown later, T_m, M_{hm}, and M_{vm}, and, hence, U (Equation 5) may be readily expressed in terms of the indeterminates, $X_0 \ldots X_n$, and the known quantities. If, in the resulting equation, the condition of statical equilibrium is introduced by putting, for X_n and X_{n-1}, their values in terms of the other $(n - 1)$ variables (using Equations 1 and 2), U will be expressed as a function of $X_0 \ldots X_{n-2}$.

Differentiating under the integral sign, we readily obtain the $(n - 1)$ partial derivatives required for finding those values of X which, consistent with statical equilibrium, will make the elastic potential a minimum. Using the same notation as before, they will be:

$$\left.\begin{aligned}
\frac{\delta U}{\delta X_0} &= \sum_{m=0}^{m=n}\left[\int_0^L \frac{T_m \delta T_m}{E F_m \delta X_0} dl + \int_0^L \frac{M_{hm} \delta M_{hm}}{E I_{hm} \delta X_0} dl\right] \\
&\quad + \sum_{m=1}^{m=n} \int_0^{a_m} \frac{M_{vm} \delta M_{vm}}{E I_{vm} \delta X_0} dy = 0 \\
\frac{\delta U}{\delta X_1} &= \sum_{m=0}^{m=n}\left[\int_0^L \frac{T_m \delta T_m}{E F_m \delta X_1} dl + \int_0^L \frac{M_{hm} \delta M_{hm}}{E I_{hm} \delta X_1} dl\right] \\
&\quad + \sum_{m=1}^{m=n} \int_0^{a_m} \frac{M_{vm} \delta M_{vm}}{E I_{vm} \delta X_1} dy = 0 \\
&\cdots\cdots\cdots\cdots \\
\frac{\delta U}{\delta X_{n-2}} &= \sum_{m=0}^{m=n}\left[\int_0^L \frac{T_m \delta T_m}{E F_m \delta X_{n-2}} dl + \int_0^L \frac{M_{hm} \delta M_{hm}}{E I_{hm} \delta X_{n-2}} dl\right] \\
&\quad + \sum_{m=1}^{m=n} \int_0^{a_m} \frac{M_{vm} \delta M_{vm}}{E I_{vm} \delta X_{n-2}} dy = 0
\end{aligned}\right\} \ldots (6)$$

In Equations (6) the first two terms correspond to the work done in the horizontal arches, and the last term to that in the vertical girder, so that these equations are of the form:

$$\frac{\delta U}{\delta X} = \frac{\delta U_h}{\delta X} + \frac{\delta U_v}{\delta X} = 0 \ldots\ldots\ldots\ldots\ldots (7)$$

The direct thrust, T, in a lock-gate is nearly constant throughout the length of the leaf for any given horizontal girder, and may be expressed very simply.

If r = the radius of a circle which passes through the centers of bearings at the quoin and miter posts when the gate is closed (Fig. 2); and

P = the load per linear unit on the horizontal girder; then

$T = P\,r$.

For any horizontal frame, m, in a gate, therefore, we can write,

$$T_m = (P_m + X_m)r. \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (8)$$

Also, if e_m is the mean eccentricity of the resultant in the horizontal girder, m, that is, the average distance between the line of pressures and the center of gravity of the cross-section, we can also write,

$$M_{hm} = e_m T_m = (P_m + X_m)\, e_m\, r. \quad \ldots\ldots\ldots\ldots\ldots\ldots (9)$$

The first two terms in Equations (6), which correspond to the work in the horizontal arches, therefore, may be written in the form:

$$\frac{\delta\, U_h}{\delta\, X} = r^2 \sum_{m=0}^{m=n} \frac{1}{E}\left[\frac{1}{F_m} + \frac{e_m^2}{I_{hm}}\right] \int_0^L \frac{(P_m + X_m)\,\delta\,(P_m + X_m)}{\delta\, X}\, d\,l\,. \quad (10)$$

Similarly, the last term of Equations (6) and (7), corresponding to work in the vertical girder, may be easily expressed as follows:

Let M_{vm} be the bending moment in any point at any panel, m, and let the distance of this point below the panel point next above $= y$. We can then write,

$$M_{vm} = L[(h_{m-1} + y)\, X_0 + (h_{m-1} - h_1 + y)\, X_1 + \ldots + (h_{m-1} - h_{m-2} + y)\, X_{m-2} + y\, X_{m-1}]. \quad \ldots\ldots (11)$$

The partial derivatives will be,

$$\frac{\delta\, M_{vm}}{\delta\, X_0} = L\,(h_{m-1} + y), \quad \frac{\delta\, M_{vm}}{\delta\, X_1} = L\,(h_{m-1} - h_1 + y)$$

$$\ldots \quad \frac{\delta\, M_{vm}}{\delta\, X_{m-1}} = L\, y.$$

That portion of the last term in Equations (6) which corresponds to the work in Panels (1) to $(n - 1)$ will then be given by the following expression:

$$\sum_{m=1}^{m=n-1} \int_0^{a_m} \frac{M_{vm}\, \delta\, M_{vm}}{E\, I_{vm}\, \delta\, X_0}\, d\, y$$

$$= \sum_{m=1}^{m=n-1} \frac{L^2}{E\, I_{vm}} \left[\left\{ h^2_{m-1}\, a_m + 2\, h_{m-1} \frac{a_m^2}{2} + \frac{a_m^3}{3} \right\} X_0 \right.$$

$$+ \left\{ h_{m-1}\, (h_{m-1} - h_1)\, a_m + (2\, h_{m-1} - h_1) \frac{a_m^2}{2} + \frac{a_m^3}{3} \right\} X_1$$

$$\left. + \quad . \quad . \quad . \quad + \left\{ h_{m-1} \frac{a_m^2}{2} + \frac{a_m^3}{3} \right\} X_{m-1} \right] \quad \ldots\ldots\ldots (12)$$

and there will be similar forms corresponding to the other variables, $X_1 \quad . \quad . \quad . \quad X_{n-2}$.

The part of the last term of Equations (6) corresponding to the work in the bottom panel must be obtained in a somewhat different way, as it is necessary to express X_n and X_{n-1} in terms of $X_0 \ldots X_{n-2}$.

Let z = the distance of any point in the panel from the bottom of the gate; then we can write:

$$M_{vn} = L\, z\, X_n$$

From Equations (1) and (2), if we take moments about the girder, $(n - 1)$,

$$X_n = \frac{1}{a_n} \left\{ h_{n-1}\, X_0 + (h_{n-1} - h_1)\, X_1 \right.$$

$$\left. + \quad . \quad . \quad . \quad (h_{n-1} - h_{n-2})\, X_{n-2} \right\}$$

hence,

$$M_{vn} = \frac{L\, z}{a_n} \left\{ h_{n-1}\, X_0 + (h_{n-1} - h_1)\, X_1 \right.$$

$$\left. + \quad . \quad . \quad . \quad (h_{n-1} - h_{n-2})\, X_{n-2} \right\} \quad \ldots\ldots\ldots\ldots (13)$$

for which we readily derive,

$$\int_0^{a_n} \frac{M_{vn}}{E\, I_{vn}} \frac{\delta\, M_{vn}}{\delta\, X_0} = \frac{L^2\, a_n}{3\, E\, I_{vm}} [h^2_{n-1}\, X_0 + h_{n-1}\, (h_{n-1} - h_1)\, X_1$$

$$+ \quad . \quad . \quad . \quad h_{n-1}\, (h_{n-1} - h_{n-2})\, X_{n-2}] \quad \ldots\ldots\ldots (14)$$

which represents the term corresponding to work in the bottom panel of the vertical girder. We can readily obtain similar forms when $X \quad . \quad . \quad . \quad X_{n-2}$ are the variables.

General Equations of Condition.—Combining Equations (10), (12), and (13), and omitting the common factor, $\frac{1}{E}$, we have the following general equations.

With X_0 as the independent variable :

$$\begin{aligned}\frac{\delta U}{\delta X_0} = r^2 \sum_{m=0}^{m=n} \left[\frac{1}{F_m} + \frac{e^2{}_m}{I_{hm}}\right] \int_0^L \frac{(P_m + X_m)\,\delta\,(P_m + X_m)}{\delta X_0}\, d\,l \\ + L^2 \sum_{m=1}^{m=n-1} \frac{1}{I_{vm}} \left[\left\{ h^2{}_{m-1}\, a_m + 2\, h_{m-1} \frac{a_m{}^2}{2} + \frac{a_m{}^3}{3} \right\} X_0 \right. \\ + \left\{ h_{m-1}\,(h_{m-1} - h_1)\, a_m + (2\, h_{m-1} - h_1) \frac{a_m{}^2}{2} + \frac{a_m{}^3}{3} \right\} X_1 \\ + \ldots + \left. \left\{ h_{m-1} \frac{a_m{}^2}{2} + \frac{a_m{}^3}{3} \right\} X_{m-1} \right] \\ + \frac{L^2\, a_n}{3\, I_{vn}} [h^2{}_{n-1}\, X_0 + h_{n-1}\,(h_{n-1} - h_1)\, X_1 \\ + \ldots + h_{n-1}\,(h_{n-1} - h_{n-2})\, X_{n-2}] = 0\end{aligned}$$

and with X_1 as the independent variable:

$$\begin{aligned}\frac{\delta U}{\delta X_1} = r^2 \sum_{m=0}^{m=n} \left[\frac{1}{F_m} + \frac{e_m{}^2}{I_{hm}}\right] \int_0^L \frac{(P_m + X_m)\,\delta\,(P_m + X_m)}{\delta X_1}\, d\,l \\ + L^2 \sum_{m=1}^{m=n-1} \frac{1}{I_{vm}} \left[\left\{ h_{m-1}\,(h_{m-1} - h_1)\, a_m \right. \right. \\ \left. + (2\, h_{m-1} - h_1) \frac{a_m{}^2}{2} + \frac{a_m{}^3}{3} \right\} X_0 \\ + \left\{ (h_{m-1} - h_1)^2\, a_m + 2\,(h_{m-1} - h_1) \frac{a_m{}^2}{2} + \frac{a_m{}^3}{3} \right\} X_1 + \ldots \\ \left. + \left\{ (h_{m-1} - h_1) \frac{a_m{}^2}{2} + \frac{a_m{}^3}{3} \right\} X_{m-1} \right] \\ + \frac{L^2\, a_n}{3\, I_{vn}} [(h_{n-1} - h_1)^2\, X_1 + \ldots + (h_{n-1} - h_1) \\ (h_{n-1} - h_{n-2})\, X_{n-2}] = 0\end{aligned} \qquad \ldots (15)$$

and equations of similar form for $X_2 \ldots X_{n-2}$ as independent variables.

By making the proper substitutions and summations in Equations (15), the $(n - 2)$ simultaneous equations may be written, and from these, the values, X_1 to X_{n-1}, are readily found.

APPLICATION OF FORMULAS TO GATES OF THE PANAMA CANAL.

77½-Foot Gate.—The general arrangement of the gate leaf is shown on Plate XVI and the photograph, Fig. 3, which represents a somewhat lower gate before the sheathing is attached.

The principal dimensions are as follows:

Clear width of lock = 110 ft.;

Height of gate = 77 ft. 6 in. from top of coping to center of bottom girder;

Central thickness = 7 ft. 0 in.;

Shape of gate: straight-backed.

There are sixteen horizontals, the spacing of which is shown on Plate XVI and Fig. 1 of Plate XVII, and is tabulated subsequently.

The cross-sections, moments of inertia, and eccentricities of lines of pressures, which are shown on Fig. 1, Plate XVII, and are also given later, are mean values for each of the horizontal arches. The moments of inertia for the vertical girder are calculated from the skin thickness of each panel and the average depth of the girder. If t_1 and t_2 are the thicknesses of the up-stream and down-stream sheathing, and d_1 and d_2 their distances from the center of gravity of the cross-sections of the girder, the moment of inertia is given by the expression:

$$I_v = L(t_1 d_1^2 + t_2 d_2^2).$$

Summarized, the data are as follows:

Length of leaf, L = 787 in.,

Radius of line of pressures, r = 880 in. (See Plate XVII, Fig. 5.)

Horizontal Arches.—Mean cross-sections:

$F_0 = F_1 = F_2 = F_3$ ..... = 100 sq. in.
$F_4 = F_5$ ..... = 120 " "
$F_6 = F_7 = F_{15}$ ..... = 140 " "
$F_8 = F_9$ ..... = 164 " "
$F_{10} = F_{11} = F_{12} = F_{13} = F_{14}$ ..... = 200 " "

Mean moments of inertia:

$I_0 = I_1 = I_2 = I_3$ ..... = 120 000 in.⁴
$I_4 = I_5$ ..... = 140 000 "
$I_6 = I_7 = I_{15}$ ..... = 156 000 "
$I_8 = I_9$ ..... = 167 000 "
$I_{10} = I_{11} = I_{12} = I_{13} = I_{14}$ ..... = 190 000 "

Mean eccentricity of line of pressure:

$e_0 = e_1 = e_2 = e_3 \ldots = 26$ in.
$e_4 = e_5 \ldots = 21$ "
$e_6 = e_7 = e_{15} \ldots = 16$ "
$e_8 = e_9 \ldots = 13$ "
$e_{10} = e_{11} = e_{12} = e_{13} = e_{14} \ldots = 11$ "

Vertical Girder.—Moments of inertia:

$I_1 = I_2 = I_3 \ldots = 1\,100\,000$ in.^{4}
$I_4 = I_5 \ldots = 1\,180\,000$ "
$I_6 = I_7 \ldots = 1\,240\,000$ "
$I_8 = I_9 = I_{10} = I_{11} \ldots = 1\,400\,000$ "
$I_{12} = I_{13} \ldots = 1\,580\,000$ "
$I_{14} = I_{15} \ldots = 1\,720\,000$ "

Vertical Panels.—

$a_1 = a_2 = a_3 = a_4 \ldots = 66$ in.
$a_5 = a_6 = a_7 = a_8 = a_9 = a_{10} = a_{11} = 60$ "
$a_{12} = a_{13} = a_{14} = a_{15} \ldots = 54$ "

Heights.—

$h_1 = 66$ in.	$h_6 = 384$ in.	$h_{11} = 684$ in.
$h_2 = 132$ "	$h_7 = 444$ "	$h_{12} = 738$ "
$h_3 = 198$ "	$h_8 = 504$ "	$h_{13} = 792$ "
$h_4 = 264$ "	$h_9 = 564$ "	$h_{14} = 846$ "
$h_5 = 324$ "	$h_{10} = 624$ "	$h_{15} = 900$ "

Water Pressure.—The water is assumed to extend to the top of the coping on the up-stream side of the gate, with a pool 9 ft. 6 in. deep below. Fig. 1, Plate XVII, shows the total water pressure and Fig. 2, Column *C*, of that plate, the load per linear foot on each horizontal arch.

Reduced to the linear inch of girder, these values are:

$p_0 = 73.34$ lb.	$p_5 = 768.25$ lb.	$p_{10} = 1\,419.35$ lb.
$p_1 = 229.15$ "	$p_6 = 898.45$ "	$p_{11} = 1\,467.9$ "
$p_2 = 386.67$ "	$p_7 = 1\,028.65$ "	$p_{12} = 1\,500$ "
$p_3 = 544.13$ "	$p_8 = 1\,158.9$ "	$p_{13} = 1\,580.7$ "
$p_4 = 665.38$ "	$p_9 = 1\,289.$ "	$p_{14} = 1\,593.75$ "
		$p_{15} = 796.87$ "

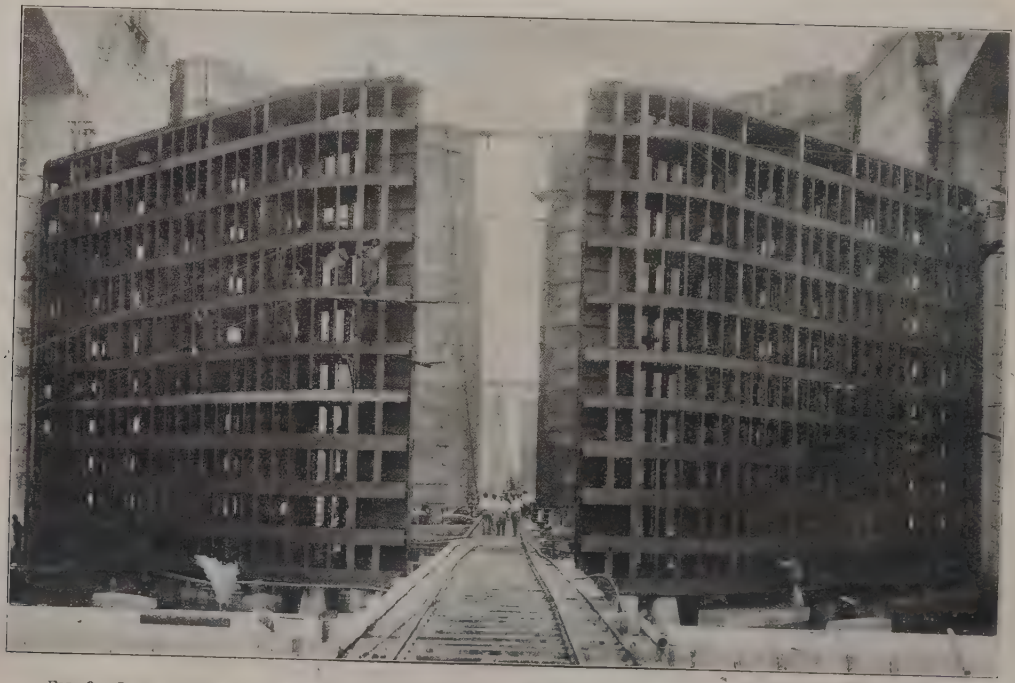


FIG. 3.—LEAVES 72 AND 73 OF PEDRO MIGUEL LOCK, PANAMA CANAL. LEAVES 68 AND 69 IN THE BACKGROUND.

The conditions of static equilibrium, as applied to the vertical girder, become:

$$X_{15} = -(X_0 + X_1 + X_2 + X_3 + X_4 + X_5 + X_6 + X_7 + X_8 + X_9 + X_{10} + X_{11} + X_{12} + X_{13} + X_{14}) \quad \ldots\ldots (16)$$

and,

$$X_{14} = -\frac{1}{54}(900\,X_0 + 834\,X_1 + 768\,X_2 + 702\,X_3 + 636\,X_4 + 576\,X_5 + 516\,X_6 + 456\,X_7 + 396\,X_8 + 336\,X_9 + 276\,X_{10} + 216\,X_{11} + 162\,X_{12} + 108\,X_{13})$$

whence,

$$X_{14} = -(16.66\,X_0 + 15.44\,X_1 + 14.22\,X_2 + 13\,X_3 + 11.78\,X_4 + 10.67\,X_5 + 9.56\,X_6 + 8.44\,X_7 + 7.33\,X_8 + 6.22\,X_9 + 5.11\,X_{10} + 4\,X_{11} + 3\,X_{12} + 2\,X_{13}) \quad \ldots\ldots (17)$$

and,

$$X_{15} = +15.66\,X_0 + 14.44\,X_1 + 13.22\,X_2 + 12\,X_3 + 10.78\,X_4 + 9.67\,X_5 + 8.56\,X_6 + 7.44\,X_7 + 6.33\,X_8 + 5.22\,X_9 + 4.11\,X_{10} + 3\,X_{11} + 2\,X_{12} + X_{13} \quad \ldots\ldots (18)$$

In order to introduce the static conditions of equilibrium, the values of X_{14} and X_{15}, in terms of $X_0 \ldots X_{13}$, as given in Equations (17) and (18), will be used as shown below.

The resulting simultaneous equations of condition, corresponding to $\frac{\delta U}{\delta X_0} = 0$, $\frac{\delta U}{\delta X_1} = 0$, etc., will then be reduced to fourteen, with $X_0 \ldots X_{13}$ as variables.

Work of Horizontal Arches.

The first terms of Equations (15) correspond to the total work of the horizontals.

Each arch must be taken up separately, and the values of the expression, $r^2\left(\frac{1}{F_m} + \frac{e^2_m}{I_{hm}}\right)\int_0^L (P_m + X_m)\frac{\delta\,(P_m + X_m)}{\delta\,X} d\,l$, obtained, taking $X \ldots X_{13}$, successively as variables. It may be convenient to represent this expression generally by the symbol H. Its values for the separate arches and variables will be the following:

Arch 0:

X_0 as variable:

$$H = 12\,106\int_0^L (P_0 + X_0)\frac{\delta\,(P_0 + X_0)\,d\,l}{\delta\,X_0} = 12\,106\,(P_0 + X_0)\,L$$
$$= 12\,106\,(78.34 + X_0)\,L = (12\,106\,X_0 + 948\,384)\,L$$

X_1 as variable:

$$H = 12\,106 \int_0^L (P_0 + X_0) \frac{\delta\,(P_0 + X_0)\,d\,l}{\delta\,X_1} = 0$$

and, similarly, we should find that $H = 0$ when $X_2 \ldots X_{13}$ are the variables.

Arch 1:

X_0 as variable:

$$H = 12\,106 \int_0^L (P_1 + X_1) \frac{\delta\,(P_1 + X_1)\,d\,l}{\delta\,X_0} = 0$$

X_1 as variable:

$$H = 12\,106 \int_0^L (P_1 + X_1) \frac{\delta\,(P_1 + X_1)\,d\,l}{\delta\,X_1} = 12\,106\,(229.15 + X_1)\,L$$
$$= (12\,106\,X_1 + 2\,774\,090)\,L.$$

For $X_2 \ldots X_{13}$, $H = 0$.

In like manner, for the arches, 3, 4 . . . 13, H will $= 0$ for every variable but one in the case of each arch. There will be a significant value, however, in the case of each arch for the one variable which has the same number as the arch in question.

These values of $H = r^2 \left(\frac{1}{F} + \frac{e^2}{I_h}\right) \int_0^L (P + X) \frac{(P + X)}{X}\,d\,l$ will be the following:

Arch 0.—X_0 as independent variable; $H = 12\,106\,X_0 + 948\,384$
Arch 1.—X_1 " " " $H = 12\,106\,X_1 + 2\,774\,090$
Arch 2.—X_2 " " " $H = 12\,106\,X_2 + 4\,681\,027$
Arch 3.—X_3 " " " $H = 12\,106\,X_3 + 6\,587\,238$
Arch 4.—X_4 " " " $H = 8\,892\,X_4 + 5\,916\,559$
Arch 5.—X_5 " " " $H = 8\,892\,X_5 + 6\,831\,279$
Arch 6.—X_6 " " " $H = 6\,802\,X_6 + 6\,111\,257$
Arch 7.—X_7 " " " $H = 6\,802\,X_7 + 6\,996\,877$
Arch 8.—X_8 " " " $H = 5\,477\,X_8 + 6\,347\,295$
Arch 9.—X_9 " " " $H = 5\,477\,X_9 + 7\,059\,853$
Arch 10.—X_{10} " " " $H = 4\,365\,X_{10} + 6\,195\,463$
Arch 11.—X_{11} " " " $H = 4\,365\,X_{11} + 6\,407\,383$
Arch 12.—X_{12} " " " $H = 4\,365\,X_{12} + 6\,547\,500$
Arch 13.—X_{13} " " " $H = 4\,365\,X_{13} + 6\,899\,755$

Arch 14.—For this arch, the general form of H will be

$$H = 4\,365 \int_0^L (P_{14} + X_{14}) \frac{\delta\,(P_{14} + X_{14})}{\delta\,X}\,dl$$
$$= 4\,365 \int_0^L (1\,593.75 + X_{14}) \frac{\delta\,(1\,593.75 + X_{14})}{\delta\,X}\,dl$$

Substituting for X_{14} its value from Equation (17), we have

$$H = 4\,365 \int_0^L (1\,593.75 - 16.66\,X_0 \ . \ . \ . \ - 2\,X_{13}) \left[\frac{1}{\delta X}\delta\,(1\,593.75 - 16.66\,X_0 \ . \ . \ . \ - 2\,X_{13})\right] d\,l.$$

For the different independent variables, we have then total values for H:

X_0 as variable:

$$H = 4\,365\ (-\ 16.66)\ (1\,593.75 - 16.66\,X_0 \ . \ . \ . \ - 2\,X_{13})\ L =$$
$$(-\ 115\,898\,930 + 1\,211\,500\,X_0 + 1\,122\,800\,X_1 + 1\,034\,000\,X_2$$
$$+\ 945\,370\,X_3 + 856\,650\,X_4 + 775\,900\,X_5 + 694\,920\,X_6$$
$$+\ 614\,100\,X_7 + 533\,330\,X_8 + 452\,500\,X_9 + 371\,700\,X_{10}$$
$$+\ 290\,883\,X_{11} + 218\,163\,X_{12} + 145\,400\,X_{13})\ L$$

X_1 as variable:

$$H = 4\,365\ (-\ 15.44)\ (1\,593.75 - 16.66\,X_0 \ . \ . \ . \ - 2\,X_{13})\ L =$$
$$(-\ 107\,411\,700 + 1\,122\,800\,X_0 + 1\,040\,590\,X_1 + 958\,400\,X_2$$
$$+\ 876\,140\,X_3 + 793\,920\,X_4 + 719\,110\,X_5 + 644\,030\,X_6$$
$$+\ 569\,090\,X_7 + 494\,280\,X_8 + 419\,330\,X_9 + 344\,460\,X_{10}$$
$$+\ 269\,580\,X_{11} + 202\,190\,X_{12} + 134\,790\,X_{13})\ L$$

and similar expressions when the differentiation is made with reference to the other independent variables, $X_2 \ . \ . \ . \ X_{13}$.

These values are not copied out in full, as the work is very voluminous. Similarly, the method is only indicated in the case of Arch 15.

Arch 15.—For this the general expression for H is:

$$H = 6\,802 \int_0^L (P_{15} + X_{15}) \frac{\delta\,(P_{15} + X_{15})}{\delta\,X}\,d\,l$$
$$= 6\,802 \int_0^L (796.87 + X_{15}) \frac{\delta\,(796.87 + X_{15})}{\delta\,X}\,d\,l.$$

If we substitute for X_{15} the value from Equation (18), we have

$$H = 6\,802 \int_0^L (796.87 + 15.66\,X_0 + 14.44\,X_1 + 13.22\,X_2$$
$$+\ 12\,X_3 + 10.78\,X_4 + 9.67\,X_5 + 8.56\,X_6 + 7.44\,X_7$$
$$+\ 6.33\,X_8 + 5.22\,X_9 + 4.11\,X_{10} + 3\,X_{11} + 2\,X_{12} + X_{13})$$
$$\times \frac{\delta\,(796.87 + 15.66\,X_0 \ . \ . \ . \ X_{13})}{\delta\,X}\,d\,l$$

which, for the different independent variables, becomes:

X_0 as variable:

$$H = 6\,802\ (15.66)\ (796.87 + 15.66\ X_0 \ldots X_{13})\ L = (84\,918\,190 + 1\,669\,518\ X_0 + 1\,539\,272\ X_1 + 1\,409\,026\ X_2 + 1\,278\,780\ X_3 + 1\,148\,534\ X_4 + 1\,030\,128\ X_5 + 911\,723\ X_6 + 793\,317\ X_7 + 674\,912\ X_8 + 556\,506\ X_9 + 438\,101\ X_{10} + 319\,695\ X_{11} + 213\,130\ X_{12} + 106\,565\ X_{13})\ L$$

and similar expressions when the differentiation is made with reference to the other independent variables, $X_1 \ldots X_{13}$.

Vertical Girder.

The second term of Equations (15) gives that part of the final equations due to the work in the vertical girder. If this term, for convenience, is called V, its value for the several panels and variables will be the following:

Panel 1.—X_0 as variable:

$$V = 0.0871\ L^2\ X_0$$

and the values for the other variables vanish.

Panel 2.—X_0 as variable:

$$V = + 0.6097\ L^2\ X_0 + 0.2175\ L^2\ X_1$$

X_1 as variable:

$$V = 0.2175\ L^2\ X_0 + 0.0871\ L^2\ X_1$$

the values for the other variables being equal to 0.

Panels 3-15.—The corresponding expressions for the remaining panels of the vertical girder will be of the same general form as those deduced for the first two panels.

In obtaining the value of V for the bottom panel (15) the variable, X, was, of course, expressed in terms of $X_0 \ldots X_{13}$.

Method of Obtaining Final Equations.

The final equations are obtained from the values for H and for V above by adding all the terms which correspond to the same independent variable, $X_0, X_1 \ldots X_{13}$, and equating them to zero.

In the case of no contact at the sill, the values of H for the bottom girder (15) must be included, as this girder then carries the full load which comes on it.

For perfect contact, the bottom girder does no work as an arch, and hence the H's for this girder must be omitted.

If the contact is such that the bottom girder does some work in arch action, a portion of its H's may be counted. In other respects, the method of calculation is independent of the degree of sill contact.

It should be noted that L (the length of the leaf, 787 in.) enters all values of H in the first power and those of V in the second. By dividing the final equations by L, it will be eliminated from the H terms, but will remain in the first power in those corresponding to V.

From their method of derivation, the resulting final equations should be symmetrical as to coefficients, and, in the equations for the solution of the problem, an average has been used where two coefficients which should be identical have differed by a slight amount. There have been no large discrepancies, proving to a considerable extent the accuracy of the work.

Only five significant figures were retained.

Final Equations of Conditions.—The final equations, as thus obtained, are the following:

(*Case A*).—*No Contact at Sill.*

X_0 as variable:

$$3\,007\,500\ X_0 + 2\,762\,700\ X_1 + 2\,530\,200\ X_2 + 2\,298\,200\ X_3 + 2\,066\,800\ X_4 + 1\,856\,900\ X_5 + 1\,647\,600\ X_6 + 1\,439\,300\ X_7 + 1\,232\,100\ X_8 + 1\,025\,700\ X_9 + 820\,580\ X_{10} + 616\,660\ X_{11} + 434\,260\ X_{12} + 252\,900\ X_{13} - 30\,032\,300 = 0.$$

X_1 as variable:

$$2\,762\,700\ X_0 + 2\,561\,000\ X_1 + 2\,335\,100\ X_2 + 2\,121\,400\ X_3 + 1\,908\,200\ X_4 + 1\,714\,800\ X_5 + 1\,521\,700\ X_6 + 1\,329\,500\ X_7 + 1\,138\,200\ X_8 + 947\,700\ X_9 + 758\,250\ X_{10} + 569\,920\ X_{11} + 401\,420\ X_{12} + 233\,890\ X_{13} - 26\,344\,200 = 0.$$

X_2 as variable:

$$2\,530\,200\ X_0 + 2\,335\,100\ X_1 + 2\,151\,900\ X_2 + 1\,944\,600\ X_3 + 1\,749\,600\ X_4 + 1\,572\,600\ X_5 + 1\,395\,800\ X_6 + 1\,219\,700\ X_7 + 1\,044\,400\ X_8 + 869\,700\ X_9 + 695\,950\ X_{10} + 523\,170\ X_{11} + 368\,580\ X_{12} + 214\,860\ X_{13} - 22\,574\,900 = 0.$$

X_3 as variable:

$$2\,298\,200\ X_0 + 2\,121\,400\ X_1 + 1\,944\,600\ X_2 + 1\,780\,000\ X_3 + 1\,591\,000\ X_4 + 1\,430\,400\ X_5 + 1\,269\,900\ X_6 + 1\,109\,900\ X_7 + 950\,550\ X_8 + 791\,700\ X_9 + 633\,630\ X_{10} + 476\,420\ X_{11} + 335\,730\ X_{12} + 195\,820\ X_{13} - 18\,806\,300 = 0.$$

X_4 as variable:

$$2\,066\,800\,X_0 + 1\,908\,200\,X_1 + 1\,749\,600\,X_2 + 1\,591\,000\,X_3 + 1\,441\,400\,X_4 + 1\,288\,300\,X_5 + 1\,144\,000\,X_6 + 1\,000\,100\,X_7 + 856\,700\,X_8 + 713\,680\,X_9 + 571\,330\,X_{10} + 429\,690\,X_{11} + 302\,890\,X_{12} + 176\,790\,X_{13} - 17\,614\,700 = 0.$$

X_5 as variable:

$$1\,856\,900\,X_0 + 1\,714\,800\,X_1 + 1\,572\,600\,X_2 + 1\,430\,400\,X_3 + 1\,288\,300\,X_4 + 1\,167\,800\,X_5 + 1\,029\,500\,X_6 + 900\,250\,X_7 + 771\,370\,X_8 + 642\,750\,X_9 + 514\,660\,X_{10} + 387\,180\,X_{11} + 273\,010\,X_{12} + 159\,470\,X_{13} - 15\,000\,500 = 0.$$

X_6 as variable:

$$1\,647\,600\,X_0 + 1\,521\,700\,X_1 + 1\,395\,800\,X_2 + 1\,269\,900\,X_3 + 1\,144\,000\,X_4 + 1\,029\,500\,X_5 + 921\,620\,X_6 + 800\,250\,X_7 + 685\,900\,X_8 + 571\,700\,X_9 + 457\,910\,X_{10} + 344\,600\,X_{11} + 243\,110\,X_{12} + 142\,120\,X_{13} - 13\,993\,300 = 0.$$

X_7 as variable:

$$1\,439\,300\,X_0 + 1\,329\,500\,X_1 + 1\,219\,700\,X_2 + 1\,109\,900\,X_3 + 1\,000\,100\,X_4 + 900\,250\,X_5 + 800\,250\,X_6 + 707\,130\,X_7 + 600\,490\,X_8 + 500\,700\,X_9 + 401\,200\,X_{10} + 302\,050\,X_{11} + 213\,200\,X_{12} + 124\,790\,X_{13} - 11\,394\,500 = 0.$$

X_8 as variable:

$$1\,232\,100\,X_0 + 1\,138\,200\,X_1 + 1\,044\,400\,X_2 + 950\,550\,X_3 + 856\,700\,X_4 + 771\,370\,X_5 + 685\,900\,X_6 + 600\,490\,X_7 + 520\,620\,X_8 + 429\,760\,X_9 + 344\,530\,X_{10} + 259\,540\,X_{11} + 183\,340\,X_{12} + 107\,470\,X_{13} - 10\,344\,600 = 0.$$

X_9 as variable:

$$1\,025\,700\,X_0 + 947\,700\,X_1 + 869\,700\,X_2 + 791\,700\,X_3 + 713\,680\,X_4 + 642\,750\,X_5 + 571\,700\,X_6 + 500\,700\,X_7 + 429\,760\,X_8 + 364\,240\,X_9 + 287\,820\,X_{10} + 217\,000\,X_{11} + 153\,440\,X_{12} + 90\,138\,X_{13} - 7\,918\,740 = 0.$$

X_{10} as variable:

$$820\,580\,X_0 + 758\,250\,X_1 + 695\,950\,X_2 + 633\,630\,X_3 + 571\,330\,X_4 + 514\,660\,X_5 + 457\,910\,X_6 + 401\,200\,X_7 + 344\,530\,X_8 + 287\,820\,X_9 + 235\,500\,X_{10} + 174\,470\,X_{11} + 123\,560\,X_{12} + 72\,819\,X_{13} - 7\,076\,810 = 0.$$

X_{11} as variable:

$$616\,660\,X_0 + 569\,920\,X_1 + 523\,170\,X_2 + 476\,420\,X_3 + 429\,690\,X_4 + 387\,180\,X_5 + 344\,600\,X_6 + 302\,050\,X_7 + 259\,540\,X_8 + 217\,000\,X_9 + 174\,470\,X_{10} + 136\,310\,X_{11} + 93\,679\,X_{12} + 55\,495\,X_{13} - 5\,158\,560 = 0.$$

X_{12} as variable:

$$434\,260\,X_0 + 401\,420\,X_1 + 368\,580\,X_2 + 335\,730\,X_3 + 302\,890\,X_4 + 273\,010\,X_5 + 243\,110\,X_6 + 213\,200\,X_7 + 183\,340\,X_8 + 153\,440\,X_9 + 123\,560\,X_{10} + 93\,679\,X_{11} + 71\,149\,X_{12} + 39\,902\,X_{13} - 3\,482\,040 = 0.$$

X_{13} as variable:

$$252\,900\,X_0 + 233\,890\,X_1 + 214\,860\,X_2 + 195\,820\,X_3 + 176\,790\,X_4 + 159\,470\,X_5 + 142\,120\,X_6 + 124\,790\,X_7 + 107\,470\,X_8 + 90\,138\,X_9 + 72\,819\,X_{10} + 55\,495\,X_{11} + 39\,902\,X_{12} + 28\,675\,X_{13} - 1\,593\,370 = 0.$$

The method used for solving these equations is given in the Appendix. The values obtained were the following:

$X_0 = +154.60$	$X_5 = -94.82$	$X_{10} = +90.18$
$X_1 = +69.59$	$X_6 = +21.62$	$X_{11} = +33.31$
$X_2 = -26.30$	$X_7 = -82.31$	$X_{12} = -11.51$
$X_3 = -127.50$	$X_8 = +37.81$	$X_{13} = +70.62$
$X_4 = -36.99$	$X_9 = -84.21$	

and, from Equations (21) and (22),

$$X_{14} = -135.1$$
$$X_{15} = +121.0$$

These values are in pounds per linear inch of horizontal arch. In Fig. 3 (*B*), of Plate XVII, they are given per linear foot. Here, as in all other cases when X is positive, the vertical girder presses against the horizontals in a down-stream direction.

(Case B).—Perfect Contact at Sill.

X_0 as variable:

$$1\,338\,000\,X_0 + 1\,223\,000\,X_1 + 1\,121\,000\,X_2 + 1\,019\,000\,X_3 + 918\,300\,X_4 + 826\,800\,X_5 + 735\,900\,X_6 + 646\,000\,X_7 + 557\,200\,X_8 + 469\,200\,X_9 + 382\,500\,X_{10} + 297\,000\,X_{11} + 221\,100\,X_{12} + 146\,300\,X_{13} - 115\,000\,000 = 0.$$

X_1 as variable:

$$1\,223\,000\ X_0 + 1\,142\,000\ X_1 + 1\,036\,000\ X_2 + 942\,400\ X_3 + 849\,300\ X_4 + 765\,000\ X_5 + 681\,100\ X_6 + 598\,100\ X_7 + 516\,000\ X_8 + 434\,600\ X_9 + 354\,300\ X_{10} + 275\,200\ X_{11} + 204\,900\ X_{12} + 135\,600\ X_{13} - 104\,600\,000 = 0.$$

X_2 as variable:

$$1\,121\,000\ X_0 + 1\,036\,000\ X_1 + 962\,800\ X_2 + 865\,400\ X_3 + 780\,300\ X_4 + 703\,200\ X_5 + 626\,300\ X_6 + 550\,200\ X_7 + 474\,800\ X_8 + 400\,000\ X_9 + 326\,200\ X_{10} + 253\,400\ X_{11} + 188\,700\ X_{12} + 124\,900\ X_{13} - 94\,240\,000 = 0.$$

X_3 as variable:

$$1\,019\,000\ X_0 + 942\,400\ X_1 + 865\,400\ X_2 + 800\,500\ X_3 + 711\,300\ X_4 + 641\,400\ X_5 + 571\,600\ X_6 + 502\,300\ X_7 + 433\,600\ X_8 + 365\,400\ X_9 + 298\,100\ X_{10} + 231\,500\ X_{11} + 172\,500\ X_{12} + 114\,200\ X_{13} - 83\,850\,000 = 0.$$

X_4 as variable:

$$918\,300\ X_0 + 849\,300\ X_1 + 780\,300\ X_2 + 711\,300\ X_3 + 651\,200\ X_4 + 579\,600\ X_5 + 516\,800\ X_6 + 454\,300\ X_7 + 392\,400\ X_8 + 330\,800\ X_9 + 269\,900\ X_{10} + 209\,800\ X_{11} + 156\,300\ X_{12} + 103\,500\ X_{13} - 76\,030\,000 = 0.$$

X_5 as variable:

$$826\,800\ X_0 + 765\,000\ X_1 + 703\,700\ X_2 + 641\,400\ X_3 + 579\,600\ X_4 + 532\,200\ X_5 + 466\,900\ X_6 + 410\,800\ X_7 + 354\,900\ X_8 + 299\,400\ X_9 + 244\,300\ X_{10} + 189\,900\ X_{11} + 141\,500\ X_{12} + 93\,720\ X_{13} - 67\,400\,000 = 0.$$

X_6 as variable:

$$735\,900\ X_0 + 681\,100\ X_1 + 626\,300\ X_2 + 571\,600\ X_3 + 516\,800\ X_4 + 466\,900\ X_5 + 423\,700\ X_6 + 367\,000\ X_7 + 317\,300\ X_8 + 267\,800\ X_9 + 218\,700\ X_{10} + 170\,000\ X_{11} + 126\,700\ X_{12} + 83\,930\ X_{13} - 60\,370\,000 = 0.$$

X_7 as variable:

$$646\,000\ X_0 + 598\,100\ X_1 + 550\,200\ X_2 + 502\,300\ X_3 + 454\,300\ X_4 + 410\,800\ X_5 + 367\,000\ X_6 + 330\,200\ X_7 + 279\,800\ X_8 + 236\,300\ X_9 + 193\,000\ X_{10} + 150\,100\ X_{11} + 111\,900\ X_{12} + 74\,150\ X_{13} - 51\,750\,000 = 0.$$

X_8 as variable:

$$557\,200\,X_0 + 516\,000\,X_1 + 474\,800\,X_2 + 433\,600\,X_3 + 392\,400\,X_4 + 354\,900\,X_5 + 317\,300\,X_6 + 279\,800\,X_7 + 247\,800\,X_8 + 204\,800\,X_9 + 167\,400\,X_{10} + 130\,300\,X_{11} + 97\,180\,X_{12} + 64\,400\,X_{13} - 44\,670\,000 = 0.$$

X_9 as variable:

$$469\,200\,X_0 + 434\,600\,X_1 + 400\,000\,X_2 + 365\,400\,X_3 + 330\,800\,X_4 + 299\,400\,X_5 + 267\,800\,X_6 + 236\,300\,X_7 + 204\,800\,X_8 + 178\,700\,X_9 + 141\,800\,X_{10} + 110\,400\,X_{11} + 82\,400\,X_{12} + 54\,620\,X_{13} - 36\,220\,000 = 0.$$

X_{10} as variable:

$$382\,500\,X_0 + 354\,300\,X_1 + 326\,200\,X_2 + 298\,100\,X_3 + 269\,900\,X_4 + 244\,300\,X_5 + 218\,700\,X_6 + 193\,000\,X_7 + 167\,400\,X_8 + 141\,800\,X_9 + 120\,500\,X_{10} + 90\,580\,X_{11} + 67\,630\,X_{12} + 44\,850\,X_{13} - 29\,360\,000 = 0.$$

X_{11} as variable:

$$297\,000\,X_0 + 275\,200\,X_1 + 253\,400\,X_2 + 231\,500\,X_3 + 209\,800\,X_4 + 189\,900\,X_5 + 170\,000\,X_6 + 150\,100\,X_7 + 130\,300\,X_8 + 110\,400\,X_9 + 90\,580\,X_{10} + 75\,090\,X_{11} + 52\,870\,X_{12} + 35\,090\,X_{13} - 21\,420\,000 = 0.$$

X_{12} as variable:

$$221\,100\,X_0 + 204\,900\,X_1 + 188\,700\,X_2 + 172\,500\,X_3 + 156\,300\,X_4 + 141\,500\,X_5 + 126\,700\,X_6 + 111\,900\,X_7 + 97\,180\,X_8 + 82\,400\,X_9 + 67\,630\,X_{10} + 52\,870\,X_{11} + 43\,940\,X_{12} + 26\,300\,X_{13} - 14\,320\,000 = 0.$$

X_{13} as variable:

$$146\,300\,X_0 + 135\,600\,X_1 + 124\,900\,X_2 + 114\,200\,X_3 + 103\,500\,X_4 + 93\,720\,X_5 + 83\,930\,X_6 + 74\,150\,X_7 + 64\,400\,X_8 + 54\,620\,X_9 + 44\,850\,X_{10} + 35\,090\,X_{11} + 26\,300\,X_{12} + 21\,870\,X_{13} - 7\,014\,000 = 0.$$

The values of the variables in these equations are:

$X_0 = +156.10$	$X_5 = -22.82$	$X_{10} = +37.12$
$X_1 = +81.23$	$X_6 = +124.1$	$X_{11} = -198.5$
$X_2 = +0.2828$	$X_7 = +14.93$	$X_{12} = -460.7$
$X_3 = -87.82$	$X_8 = +135.9$	$X_{13} = -837.5$
$X_4 = +27.67$	$X_9 = -38.78$	

and, substituting in Equations (21) and (22),

$$X_{14} = -1\,208.0$$
$$X_{15} = +2\,276.0$$

all these values being pounds per linear inch. In Fig. 3 (*A*), of Plate XVII, these values are given in pounds per linear foot.

49½-Foot Gate.—This gate consists of the nine panels of the 77½-ft. gate which are nearest the top, but the bottom girder, (9), is somewhat modified, so that $F_9 = 120$ sq. in.; $I_9 = 140\,000$ in.^{4}; $e_9 = 21$ in.: and $P_9 = 622.8$ lb. per lin. in.

The static equations for equilibrium give:

$$X_8 = -(9.4\,X_0 + 8.3\,X_1 + 7.2\,X_2 + 6.1\,X_3 + 5\,X_4 + 4\,X_5 + 3\,X_6 + 2\,X_7)$$
$$X_9 = +(5.4\,X_0 + 7.3\,X_1 + 6.2\,X_2 + 5.1\,X_3 + 4\,X_4 + 3\,X_5 + 2\,X_6 + X_7)$$

The equations of conditions become:

(*Case A*).—*No Contact at Sill.*

$$1\,152\,900\,X_0 + 996\,270\,X_1 + 851\,980\,X_2 + 708\,100\,X_3 + 564\,850\,X_4 + 435\,260\,X_5 + 306\,440\,X_6 + 178\,480\,X_7 - 12\,197\,513 = 0.$$
$$996\,270\,X_0 + 882\,620\,X_1 + 744\,830\,X_2 + 619\,360\,X_3 + 494\,280\,X_4 + 381\,050\,X_5 + 268\,440\,X_6 + 156\,550\,X_7 - 9\,481\,511 = 0.$$
$$851\,980\,X_0 + 744\,830\,X_1 + 649\,790\,X_2 + 530\,590\,X_3 + 423\,710\,X_4 + 326\,850\,X_5 + 230\,440\,X_6 + 134\,610\,X_7 - 6\,684\,281 = 0.$$
$$708\,100\,X_0 + 619\,360\,X_1 + 530\,590\,X_2 + 453\,960\,X_3 + 353\,120\,X_4 + 272\,640\,X_5 + 192\,440\,X_6 + 112\,660\,X_7 - 3\,887\,777 = 0.$$
$$564\,850\,X_0 + 494\,280\,X_1 + 423\,710\,X_2 + 353\,120\,X_3 + 291\,460\,X_4 + 218\,430\,X_5 + 154\,440\,X_6 + 90\,720\,X_7 - 3\,668\,164 = 0.$$
$$435\,260\,X_0 + 381\,050\,X_1 + 326\,850\,X_2 + 272\,640\,X_3 + 218\,430\,X_4 + 178\,040\,X_5 + 119\,890\,X_6 + 70\,775\,X_7 - 1\,944\,087 = 0.$$
$$306\,440\,X_0 + 268\,440\,X_1 + 230\,440\,X_2 + 192\,440\,X_3 + 154\,440\,X_4 + 119\,890\,X_5 + 92\,153\,X_6 + 50\,828\,X_7 - 1\,854\,752 = 0.$$
$$178\,480\,X_0 + 156\,550\,X_1 + 134\,610\,X_2 + 112\,660\,X_3 + 90\,720\,X_4 + 70\,750\,X_5 + 50\,828\,X_6 + 37\,683\,X_7 - 159\,775 = 0.$$

The values of the variables become:

$X_0 = +148.71$	$X_3 = -139.92$	$X_6 = +9.32$	$X_9 = +147.72$
$X_1 = +61.24$	$X_4 = -53.57$	$X_7 = -82.38$	
$X_2 = -36.48$	$X_5 = -110.26$	$X_8 = +55.72$	

(Case B).—Perfect Contact at Sill.

$$525\,440\,X_0 + 451\,010\,X_1 + 388\,880\,X_2 + 327\,170\,X_3 + 266\,080\,X_4 + 211\,180\,X_5 + 157\,050\,X_6 + 103\,790\,X_7 - 58\,716\,000 = 0.$$

$$451\,010\,X_0 + 408\,770\,X_1 + 342\,380\,X_2 + 288\,310\,X_3 + 234\,630\,X_4 + 186\,320\,X_5 + 138\,620\,X_6 + 91\,640\,X_7 - 49\,938\,000 = 0.$$

$$388\,880\,X_0 + 342\,380\,X_1 + 307\,990\,X_2 + 249\,430\,X_3 + 203\,190\,X_4 + 161\,460\,X_5 + 120\,180\,X_6 + 79\,476\,X_7 - 41\,019\,000 = 0.$$

$$327\,170\,X_0 + 288\,310\,X_1 + 249\,430\,X_2 + 222\,680\,X_3 + 171\,730\,X_4 + 136\,590\,X_5 + 101\,740\,X_6 + 67\,314\,X_7 - 32\,131\,000 = 0.$$

$$266\,080\,X_0 + 234\,630\,X_1 + 203\,190\,X_2 + 171\,730\,X_3 + 149\,180\,X_4 + 111\,720\,X_5 + 83\,305\,X_6 + 55\,154\,X_7 - 25\,820\,000 = 0.$$

$$211\,180\,X_0 + 186\,320\,X_1 + 161\,460\,X_2 + 136\,590\,X_3 + 111\,720\,X_4 + 98\,020\,X_5 + 66\,544\,X_6 + 44\,099\,X_7 - 18\,558\,000 = 0.$$

$$157\,050\,X_0 + 138\,620\,X_1 + 120\,180\,X_2 + 101\,740\,X_3 + 83\,305\,X_4 + 66\,544\,X_5 + 56\,585\,X_6 + 33\,044\,X_7 - 12\,931\,000 = 0.$$

$$103\,790\,X_0 + 91\,640\,X_1 + 79\,476\,X_2 + 67\,314\,X_3 + 55\,154\,X_4 + 44\,099\,X_5 + 33\,044\,X_6 + 28\,791\,X_7 - 5\,697\,700 = 0.$$

The values of the variables become:

$X_0 = +310.1$	$X_3 = -96.30$	$X_6 = -301.4$	$X_8 = +1\,615.6$
$X_1 = +184.4$	$X_4 = -74.07$	$X_7 = -593.4$	
$X_2 = +52.4$	$X_5 = -225.5$	$X_8 = -871.9$	

Discussion of Results.

The values of the variables, X_0, X_1, etc., previously obtained, give the reactions between the horizontal and vertical girders for the two gates and different conditions of sill contact, and $(P_0 + X_0)$, $(P_1 + X_1)$, etc., are the resultant girder loads per linear unit of leaf. Plate XVII gives these results in graphic form for the 77 ft. 6-in. and 49 ft. 6-in. gates, respectively.

On this plate, Figs. 1 and 6 show cross-sections of the leaf with its dimensions, moments of inertia, etc., also the total water pressure acting against the gate, and Figs. 3 and 8 give the reactions between the horizontals and the vertical girder in pounds per linear foot of gate leaf, *B* being for no contact and *A* for perfect contact at sill.

Figs. 2 and 7 of Plate XVII give the resultant loads per linear foot on the different horizontals for the three different cases: *A*, vertical

stiffness and perfect sill contact; *B,* vertical stiffness but no sill contact; and *C,* no vertical stiffness at all.

Figs. 4 and 9 of Plate XVII show the deflections of the miter posts, that is, the distance they move down stream parallel with the axis of the lock for the three cases just mentioned. These last curves are in general agreement with those showing the resultant girder loads.

Loads on Horizontals.—The curves show that, with no contact at the sill, the values of X (reactions of vertical against horizontal girders) are quite small, so that, except at the very top, the deviations from a purely hydrostatic loading are inconsiderable and due to accidental causes. With "perfect" contact, the girders in the lower portion of the leaf (from one-third to one-half the height) were relieved of a large part of their hydrostatic load, the horizontals higher up receiving a proportionately greater loading. The girders closest to the top showed, in all cases, the largest proportional increase. In the 77 ft. 6-in. gate, there was also an increase for the horizontals in the middle third of the height. As a whole, however, the effect of the vertical stiffness was decidedly greater for the 49 ft. 6-in. gate.

In proportioning the Panama gates, it was decided to use a load corresponding to a head of 20 ft. for all girders within 20 ft. of the top. For those lower down, the hydrostatic head was taken, but, in gates more than 75 ft. high, the girders in the middle third of the height had this load increased by from 5 to 10 per cent.

For smaller gates, the effect of vertical stiffness would probably be greater than for the very high and rather thin Panama gates.

There is no reason, however, to doubt that the common assumption of hydrostatic loading, except for a few girders near the top, will give safe results. In the lower part of the leaf, the stresses in actual service will generally be quite small in a gate designed for the hydrostatic head, as it is very probable that there will be some contact at the sill. However, the increase in cost involved is not great, and in any event the miter gate will weigh much less than any form of caisson or single-leaf gate.

Reaction of Gate Against Sill.—For the case of no contact, the reaction, of course, is equal to zero. For "perfect contact", the sill pressure is equal to the end reaction, X, at the bottom of the vertical girder plus the direct water load, P, on the lowest arch.

These values for the two gates are as follows: (See Figs. 2 and 7 of Plate XVII).

For the 77 ft. 6-in. gate,

$$12\,(P_{15} + X_{15}) = 9\,560 + 27\,322 = 36\,882 \text{ lb. per lin. ft.}$$

or, $$\frac{36\,882}{184\,875} = 19.4\% \text{ of the total load of the gate.}$$

For the 49 ft. 6-in. gate,

$$12\,(P_9 + X_9) = 7\,474 + 19\,387 = 26\,857 \text{ lb. per lin. ft.}$$

or, $$\frac{26\,857}{76\,565} = 35.1\% \text{ of the total load.}$$

It will be noted that the total sill reaction is proportionately greater for the lower gate.

In case there is some elastic movement of the sill under pressure, the sill reaction and also the loads on horizontals and the deflections will be values intermediate between Cases *A* and *B*.

As stated previously, it is believed that Case *A* is a sufficient maximum.

For small locks, it is not unusual to make the masonry sill strong enough to withstand the theoretical maximum of 66⅔% of the total water pressure acting against the gate. For the large proportions at Panama, it would have been difficult to make the sill walls strong enough to carry this maximum, and it seemed entirely unnecessary. It was deemed quite safe to assume a pressure of 50 000 lb. per lin. ft. of sill or about one-quarter of the whole load on the gate.

Stresses in the Vertical Bracing.—This bracing corresponds to the vertical girder in the computations, the loads acting on it being the forces, X_0, X_1, etc., applied transversely at distances corresponding to the spacing of the horizontal girders.

The chord stresses are readily obtained from the values shown on Plate XVII. The unit stresses were found to be not more than 4 500 lb. per sq. in. in any part of the bracing.

The shears for proportioning the web thickness and the rivet connections were obtained in a similar manner from the transverse forces, X_0, X_1, etc.

APPENDIX

Method for the Solution of Simultaneous Equations.

For the solution of simultaneous equations of the first degree, the method originated by the celebrated astronomer, Gauss, is probably the best for practical use in the engineer's office. The solution by determinants is theoretically attractive, but its application is not satisfactory. Graphical methods are the best in some cases, but can hardly be used where the coefficients of the different variables differ widely in magnitude, and where a high degree of accuracy is required.

Gauss' method in a simple form may be stated as follows:

Given, say, three simultaneous equations of the form:

$$a\,x + b\,y + c\,z + m = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots (1)$$
$$b\,x + d\,y + e\,z + n = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots (2)$$
$$c\,x + e\,y + f\,z + p = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

to find the values of x, y, and z.

From Equation (1), we obtain:

$$x = -\frac{b\,y + c\,z + m}{a}.$$

and substituting in Equations (2) and (3), we have:

$$\left(d - \frac{b}{a}\,b\right) y + \left(e - \frac{b}{a}\,c\right) z + \left(n - \frac{b}{a}\,m\right) = 0 \ldots\ldots (2^1)$$

$$\left(e - \frac{c}{a}\,b\right) y + \left(f - \frac{c}{a}\,c\right) z + \left(p - \frac{c}{a}\,m\right) = 0 \ldots\ldots (3^1)$$

which may be written,

$$d_1\,y + e_1\,z + n_1 = 0 \quad (2^1)$$
$$e_1\,y + f_1\,z + p_1 = 0 \quad (3^1)$$

In these two equations the coefficients are the same as in Equations (2) and (3), with a subscript added.

From Equations (2^1) and (3^1), we have, in the same way,

$$\left(f_1 - \frac{e_1}{d_1}\,e_1\right) z + \left(p_1 - \frac{e_1}{d_1}\,n_1\right) = 0 \quad \ldots\ldots\ldots\ldots (3^2)$$

which may be written,

$$f_2\,z + p_2 = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots (3^2)$$

From Equation (3^2), we obtain the value of $z = -\frac{p_2}{f_2}$, and the value of x and y by substituting in the first and second sets of equations.

By writing the equation in tabular form, the relation of the successive coefficients is made clearer, and a valuable check on the arithmetical work is obtained at each step.

TABLE 1.

	x	y	z	—	—
Equation (1)	a	b	c	m	$q = a + b + c + m$
Equation (2)	b	d	e	n	$r = b + d + e + n$
Equation (3)	c	e	f	p	$s = c + e + f + p$

TABLE 2.

—	y	z	—	—
Equation (2^1)	d_1	e_1	n_1	r_1
Equation (3^1)	e_1	f_1	p_1	s_1

TABLE 3.

—	z	—	—
Equation (3^2)	f_2	p_2	s_2

On examining Equations (2^1) and (3^1), it will be seen that the coefficients, d_1, e_1, etc., are always of the form,

$$T - \frac{U\,V}{H},$$

in which,

T = the corresponding coefficient in Equations (2) and (3), that is, the coefficient of the same variable, omitting the subscript;

U = the coefficient in the top horizontal line, that is, in Equation (1) vertically above T;

V = the last coefficient to the left in the same horizontal line with T; and

H = the coefficient of x in the top row.

The values of q, r, and s, in the last column of Table 1 are written down by simply adding the preceding coefficients in each horizontal row.

In Tables 2 and 3, r_1, s_1, s_2, like the coefficients, d_1, e_1, n_1, etc., are obtained by the formula, $T - \frac{U\,V}{H}$, previously given.

The check consists in adding up each horizontal line; the last term should be equal to the sum of the preceding ones, that is, for instance, s should equal $e_1 + f_1 + p_1$.

Numerical Example.

$$482.5\,x + 348.4\,y + 238.6\,z + 140.7\,u - 48\,915 = 0$$
$$348.4\,x + 280.5\,y + 186.0\,z + 111.4\,u - 37\,167 = 0$$
$$238.6\,x + 186.0\,y + 146.9\,z + 82.0\,u - 24\,484 = 0$$
$$140.7\,x + 111.4\,y + 82.0\,z + 66.0\,u - 11\,760 = 0$$

TABLE 4.

x	y	z	u	——	——
482.5	348.4	238.6	140.7	− 48 915	− 47 704.8
348.4	280.5	186.0	111.4	− 37 167	− 36 240.1
238.6	186.0	146.9	82.0	− 24 484	− 23 330.5
140.7	111.4	82.0	66.0	− 11 760	− 11 359.9

TABLE 5.

y	z	u	——	——
28.92	13.70	9.80	− 1 847	− 1 794.5
13.70	28.90	12.42	− 296	− 2 410.0
9.80	12.42	24.97	+ 2 503	+ 2 550.1

TABLE 6.

z	u	——	——
22.41	7.78	+ 579	+ 609.2
7.78	21.65	+ 3 128.9	+ 3 158.3

TABLE 7.

	u	——	——
	18.95	+ 2 927.9	+ 2 946.84

The coefficients for the first equation in Table 5, will be,

$$280.5 - \frac{348.4 \times 348.4}{482.5} = 28.92$$

$$186.0 - \frac{238.6 \times 348.4}{482.5} = 13.70$$

$$111.4 - \frac{140.7 \times 348.4}{482.5} = 9.80$$

$$-37\,167 - \frac{48\,915 \times 348.4}{482.5} = -1\,847$$

and, as a check, the last term

$$\left(-36\,240.1 - \frac{47\,704.8 \times 348.4}{482.5}\right) = -1\,794.5$$

which should and does equal the sum of the preceding terms.

The roots are $x = +58$, $y = +103$, $z = +27$, and $u = -154$.

The computations were carried out mainly with the use of the well-known cylindrical slide-rule of Edwin Thacher, M. Am. Soc. C. E., and the "Millionaire" multiplying machine.

Although such operations are necessarily somewhat tedious, the set of equations containing fourteen variables was solved by two computers in less than 15 hours of actual work.

MEMOIRS OF DECEASED MEMBERS

MENDES COHEN, Past-President, Am. Soc. C. E.*

DIED AUGUST 13TH, 1915.

Mendes Cohen was born in Baltimore, Md., on May 4th, 1831. His family, previously prominent in Europe, came to America before the Revolution, its first representative going to Lancaster, Pa., in 1773, afterward serving in the war and, later, living in Richmond, Va. Mr. Cohen's grandfather went to Baltimore in 1808. Two of the family served in the War of 1812. The father of Mendes Cohen, a banker in Baltimore, died in 1847, and the son, who had been under the instruction of a private tutor, directly entered the works of Ross Winans, the builder of early American locomotives. In 1851, he was made an Assistant Engineer on the Baltimore and Ohio Railroad, and served on the construction of the Broad Tree Tunnel; then, having been transferred to the Motive Power Department, he was engaged in the adaptation of wood-burning passenger locomotives to coal burning. He also worked out the method adopted for handling the traffic on the 10% temporary grade over the Kingwood Tunnel, a remarkable achievement in railroad operation.

In 1855, when only twenty-four years old, Mr. Cohen had already become known as an especially capable railroad official, and was made Assistant Superintendent of the Hudson River Railroad. He was with that Company until 1861, when he succeeded Gen. George B. McClellan as operating head of the Ohio and Mississippi Railroad, first as Superintendent and, later, as President and Superintendent. Soon after the close of the Civil War, he was engaged, for a short time, on special work for the Philadelphia and Reading Railway.

From 1868 to 1871 Mr. Cohen was Comptroller and Assistant to the President of the Lehigh Coal and Navigation Company. In 1872, he became President of the Pittsburgh and Connellsville Railroad, which was subsequently consolidated with the Baltimore and Ohio System. He retired in 1875 from official positions in connection with companies, but continued his practice in Baltimore as a Consulting Engineer.

Mr. Cohen is survived by his widow, Mrs. Jessie (Nathan) Cohen.

Mr. Cohen was a member of the Board appointed by the President, in 1894, to report on a route for the Chesapeake and Delaware Ship Canal. From 1893 to 1904, he was Chairman of the Baltimore Sewer-

* Memoir prepared by the following Committee: John Bogart and J. E. Greiner, Members, Am. Soc. C. E.

age Commission. He was for twenty-one years the Secretary of the Maryland Historical Society, serving for nine years as its President. He was also Vice-President of the American Jewish Historical Society from 1897 until his death. His acquaintance with history, especially that of Maryland, was remarkable. He was a member of the Municipal Art Commission of Baltimore from 1892 until his death. For many years he was also an active member of the Board of Trustees of the Peabody Institute.

During many years Mr. Cohen was a strong and influential citizen in the city of his birth and long residence. He was consulted in all matters of great importance, and his judgment, always given after deliberate consideration, had great weight. He was a serious, religious man; socially of the best in Baltimore, in fact, a worthy representative of one of the old strong families of the United States.

Mr. Cohen was a man of strong character, with broad experience, not only in technical and engineering matters, but also in well-informed studies of affairs and conditions of business and social life. He was a man of much reserve, wise in conference, ready to consider and give weight to the suggestions of others, and did not hesitate to express his own judgment and opinion with clear statements of reasons for them. He was, therefore, a strong and valuable associate and adviser in the direction of business matters. This was particularly evident in the consideration of the problems arising in the affairs of a growing institution, such as the Society was during the years of his close connection with its management. He never lost interest in its welfare, nor forgot the responsibilities of a Past-President. He highly appreciated the honor of his election as its President.

Mr. Cohen became a Member of the American Society of Civil Engineers on December 4th, 1867. The Society, founded in 1852, had been inactive during the Civil War, but was kept alive by a small group of members, the officers elected in 1852 continuing their nominal functions. In 1867, the Society was resuscitated, James P. Kirkwood, one of the original members, becoming President in succession to James Laurie, who had held the office from 1852 to 1867. Mr. Cohen was elected to membership in that year, and, at the time of his death, had only one living associate of those who became members in 1867. The Society had only twenty-six members at the time of his election.

Although then only thirty-six years old, Mr. Cohen had already held responsible positions in railroad operation and management and his well-informed judgment was at once of value in the building up and judicious expansion of the activities of the Society. He served as a Director in 1888, as Vice-President in 1890, and as President in 1892.

ELMER LAWRENCE CORTHELL, President, Am. Soc. C. E.*

DIED MAY 16TH, 1916.

Elmer Lawrence Corthell was born in South Abington (now Whitman), Mass., in 1840, the son of James Lawrence and Mary Gurney Corthell. He was educated at the South Abington High School and Phillips Exeter Academy. He entered Brown University in 1859, studying until the outbreak of the Civil War in 1861. At the first call for volunteers, he enlisted in the First Rhode Island Artillery, Battery D, and served in stations from private to the rank of Commander of field artillery, until his return with rank of Captain at the end of the war. As a soldier, he saw service principally in Virginia and North Carolina.

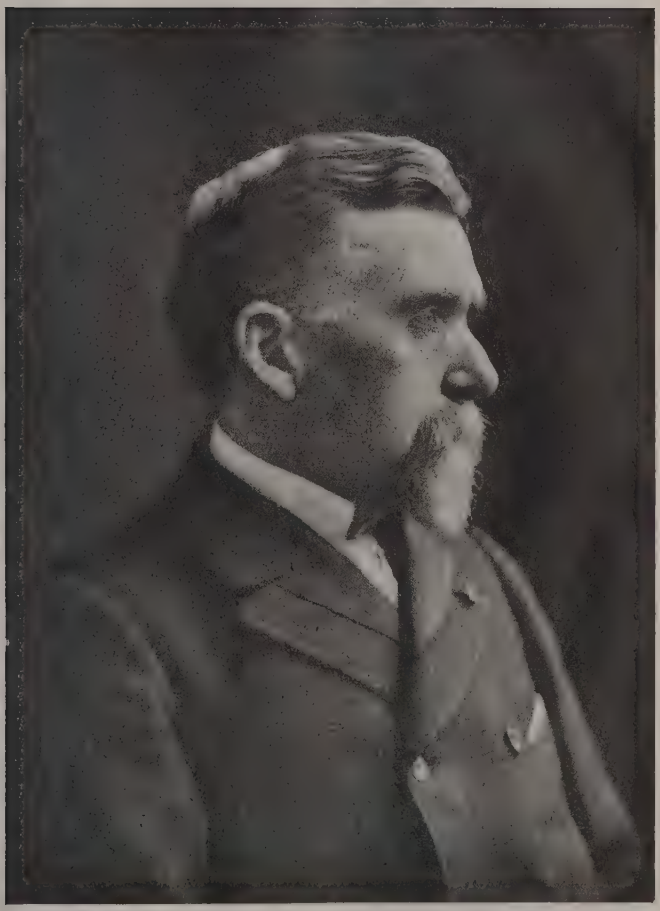
At the close of the war Mr. Corthell returned and resumed his studies at Brown University from which he was graduated in 1867 with the degree of Master of Arts, and Phi Beta Kappa honors.

In 1894 he received from his Alma Mater the honorary degree of Doctor of Science. From boyhood it had been Mr. Corthell's aim and ambition to study for the ministry in the Baptist Church, but on completing his university course, on account of his health, and the advice of his physician to select a more active profession, he chose Civil Engineering and entered the employ of the late Samuel Barrett Cushing, M. Am. Soc. C. E., a prominent engineer in Providence, R. I., where he practiced and did field work in railroad, mill, dam, bridge, city, and other construction.

In 1868 Mr. Corthell was appointed Assistant Engineer on the construction of the Hannibal and Naples Railroad in Illinois, now a part of the Wabash System. In 1869, he was Engineer on the location and construction of the Hannibal and Missouri Railroad. In 1870-71, he was Chief Assistant Engineer, under the late Col. E. D. Mason, M. Am. Soc. C. E., in the construction of the railroad bridge over the Mississippi River at Hannibal, Mo. From 1871 to 1875, he was Chief Engineer of the Sny Island Levee, 51 miles in length, on the east bank of the Mississippi River. While engaged in that work he was, in 1873-75, Chief Engineer on the construction of the bridge over the Mississippi River, at Louisiana, Mo., for the Chicago and Alton Railroad. The draw-span of this bridge was 444 ft. long—at that time, the longest in the world.

In July, 1875, Mr. Corthell went to the mouth of the Mississippi River, and from that time until 1880 was Assistant to the late Capt. James B. Eads, M. Am. Soc. C. E., in the construction of the jetties

* Memoir prepared by the following committee: John F. Wallace and J. A. Ockerson, Past-Presidents, Am. Soc. C. E., and W. J. Karner, Assoc. Am. Soc. C. E.



C. L. Corthell

at the South Pass mouth of the river. On account of ill-health, he came north early in 1880, and while convalescing, wrote his "History of the Jetties at the Mouth of the Mississippi River".*

From 1881 to 1884, Mr. Corthell served as Engineer of the New York, West Shore and Buffalo Railroad Company, in charge of the construction of that road and the extension of the New York, Ontario and Western Railroad to connect with the West Shore. For about two years, during the construction of the West Shore Railroad, he was assisting Capt. Eads and Col. James Andrews in their plans for the construction of a ship railway across the Isthmus of Tehuantepec, Mexico, and, after he resigned his position with the West Shore Railroad Company, from 1884-87, he assisted Capt. Eads in promoting the Tehuantepec Ship Railway, taking charge, on the Isthmus, of the survey of the route for the railway, and exploiting it by visiting various cities in the United States, showing perfect working models of the railroad and a ship being placed in its cradle on the rails.

From March, 1887, until 1889, Mr. Corthell was associated in an engineering partnership, in New York City and Chicago, Ill., with the late George S. Morison, Past-President, Am. Soc. C. E., engaged in the construction of railroads, bridges, harbors, and water-works. During this partnership there were constructed, the Cairo Bridge over the Ohio River for the Illinois Central Railroad, then the longest steel bridge in the world, bridges over the Missouri River at Nebraska City and Sioux City, two bridges in Oregon, one at Jacksonville, Fla., and water-works at Bismarck, N. Dak.

From 1889 to 1890, he was Engineer of the St. Louis Merchants Bridge over the Mississippi River, having charge of the design and construction of the substructure and foundations.

During the same period, from 1889 to 1890, he was also Chief Engineer of the improvements at the mouth of the Brazos River, in Texas.

In 1889 and 1890 he was employed as a special Consulting Engineer in charge of terminal work in the City of Chicago for the Chicago, Madison and Northern Railroad, a subsidiary corporation of the Illinois Central Railroad,—the Chicago, Madison and Northern Railroad affording an entrance to Chicago from the West for the Illinois Central lines; and also in connection therewith for certain main and side-track facilities for the Atchison, Topeka and Santa Fe and the Chicago and Alton Railroads.

In 1889, Mr. Corthell made examinations, plans, and report on the proposed improvement of the harbor at Tampico, Mexico, for the Mexican Central Railroad and, later, had charge of the construction of the jetties as Chief Engineer.

During 1889 he was also President and Chief Engineer of the Southern Bridge and Railway Company, incorporated that year to build a

* Published in 1880 by John Wiley and Sons.

bridge over the Mississippi River, at New Orleans, and completed the plans and specifications for its construction.

In the following year (1890), he made a personal examination between the Great Lakes and Quebec, Canada, of the question of an enlarged waterway between Chicago, Duluth, and other ports of the Great Lakes and the Atlantic Seaboard, and wrote a paper on this subject for the Canadian Society of Civil Engineers* and the Western Society of Engineers at Chicago.

In 1891 Mr. Corthell visited Europe, with several important objects in view. As a Trustee of the University of Chicago, he examined six of the leading universities and technical schools of Europe in order to obtain information for the University in carrying out its purpose of establishing, in connection with it, a great School of Engineering and Architecture. As a member of a Committee of the Western Society of Engineers, engaged in solving the difficult railroad problem of Chicago, he examined in Europe thirty-five railroad terminals and complicated situations. He also examined twenty-six European harbors to secure special information for use in connection with his work at Tampico, Mexico, and elsewhere. He also examined nearly all the subways of the Old World, from Glasgow to Budapest.

In 1897, Dr. Corthell again visited Europe to examine a great variety of engineering works. Many of the results of his various examinations and investigations were published in the *Engineering Magazine,* in New York and London. The most extensive work done by him, however, during his two years in Europe, was on the subject of Maritime Commerce, its past, present, and future. In August, 1898, he presented the results of his work to the American Association for the Advancement of Science, which held its Fiftieth Anniversary, at Boston, Mass. The object of the paper was to show the development of commerce in the half century to come. On his return to the United States he was engaged as expert on several important works in the United States and Mexico.

Dr. Corthell sailed for Buenos Aires, Argentine Republic, in March, 1900, where, for more than two years, he was engaged in solving problems for commerce and reporting to the Minister of Public Works. Thirty-six different subjects were referred to him for investigation and report.

During the winter of 1902 and the spring of 1903, he delivered thirty-six lectures in thirty cities of the United States and Mexico, on "Two Years in Argentine as Consulting Engineer of National Public Works". These were delivered before universities and commercial bodies, also engineering societies, etc., at the request of the Argentine Government.

* *Transactions,* The Canadian Society of Civil Engineers, Vol V, p. 32 (1891).

At one time, Dr. Corthell made an examination, reports, and estimates for the Boston, Cape Cod, and New York Ship Canal.

The Governor of the State of New York, in 1894, appointed Dr. Corthell upon the Advisory Board of Consulting Engineers on the State Barge Canal, a position from which he resigned on account of his Brazilian work.

During 1904 and 1905 he was engaged on extensive commercial works in Brazil, at Para, in St. Catherina, and Rio Grande do Sul.

He represented the United States on the Permanent Commission of the International Congress of Navigation and was an active and influential member of that organization.

Mr. Corthell was married, in 1870, to Emily Theodate Davis, of Providence, R. I., who died in 1884, leaving two children, Mrs. E. S. Dewey, of Gloversville, N. Y., and Howard Lawrence Corthell, a Civil Engineer, of New York City. In 1900, he was married to Marie Kuchler, of Berne, Switzerland, who survives him. He also leaves one brother, Roland, of Boston, and one sister, Mrs. Annie C. Phipps, of Wollaston, Mass.

The various International Engineering Congresses always found Dr. Corthell an active participant, either in person or by important papers. At a meeting in Brussels, where all but six of the seventy-one papers were in other languages than English, he prepared for the Department of State, which had selected him as a delegate, a résumé of the entire proceedings, which formed a volume of 245 pages. The International Engineering Congress held during the Columbian Exposition was suggested by him, and its success was largely due to his work as Chairman of the Executive Committee having charge of that affair. When the recent Pan-American Scientific Congress, in Washington, D. C., was contemplated, the Department of State again called upon Dr. Corthell to aid in the organization of the meeting, and he rendered valuable assistance in making this affair a success.

After years of exceedingly active work, Dr. Corthell found his chief satisfaction in the fact that his works had been beneficial to commerce by sea, river, canal, and rail, and he could point with pride to the results as having aided in reducing the cost of transportation on land and water, to the benefit of mankind.

A prolific writer on engineering subjects, his printed papers fill many volumes. Another of his activities concerned the establishment of a civilian reserve corps of engineers, and he was one of the leaders, if not the leader, in this project. After his election to the Presidency of the American Society of Civil Engineers, Dr. Corthell had taken an active part in the proposal to have the Society join the three other National Engineering Organizations in the United Engineering Societies Building in New York.

Dr. Corthell was a member of the following societies: American Institute of Consulting Engineers, Inc., of which he served as President in 1915-16; The Canadian Society of Civil Engineers; The Institution of Civil Engineers of Great Britain; The Society of Arts of Great Britain; Member d'Honneur, and Corresponding Member of the French Society of Civil Engineers; The American Association of Civil Engineers and Architects; American Railway Engineering Association; The Boston Society of Civil Engineers; The Western Society of Civil Engineers, Chicago, Ill., of which he was President in 1889; Honorary Member of the Geographical and Statistical Society of Mexico; American Geographical Society; The National Geographical Society, Washington, D. C.; Fellow, Royal Geographical Society, London, England; Fellow, American Association for the Advancement of Science; Honorary Member, Engineering Society of Portugal, The Institution of Engineers of the River Plate, The Centre de Navigacion Transatlantica, and Sociedad Cientifica of Argentine; Franklin Institute, Philadelphia, Pa.; American Highway Association; Pan-American Society; Founder, Pan-American Chamber of Commerce: Member, Chamber of Commerce of the United States; and Member, Board of Consulting Engineers, Barge Canals, New York State.

Dr. Corthell was also a member of the following military and patriotic associations: Grand Army of the Republic; Military Order of the Loyal Legion; Sons of the American Revolution; New England Society; and Society of the Army of the Potomac.

He was also a member of several academical and university societies and clubs, including the University Club, New York City, Phi Beta Kappa and Sigma Chi Societies, and of the Engineers Club of Rio de Janeiro.

After a half century of continuous work devoted to the service of his fellow-man and the advancement and up-building of the Engineering Profession, he has now passed away, leaving the Profession and the world better and enriched by his work, which has been a shining example of an enthusiastic, industrious, useful life.

Throughout it all, in adversity as well as in prosperity, he met his daily problems with enthusiasm, and ever had his hand extended to encourage and help all with whom he came in contact, particularly the younger members of the Profession. He was always ready to assist them, not only by advice and counsel, but by financial aid and the use of his influence, in obtaining positions and advancements.

His memory will always flourish in the hearts of those who have been favored by his acquaintance.

Dr. Corthell was elected a Member of the American Society of Civil Engineers, on September 2d, 1874. He served two terms as Vice-President, in 1889 and 1893-94, and was elected President on January 19th, 1916.

CHARLES CONRAD SCHNEIDER, Past-President, Am. Soc. C. E.*

DIED JANUARY 8TH, 1916.

Charles Conrad Schneider, the son of Julius and Emilie Schneider, was born in Apolda, Saxony, on April 24th, 1843. He attended school in his native city, and then served an apprenticeship in a machine shop. After this he attended the Royal School of Technology in Chemnitz, Saxony, from which he was graduated in 1864. He afterward devoted a few years to active professional practice as a Mechanical Engineer.

In 1867, Mr. Schneider came to the United States and, for three years, was employed as a Draftsman with the Rogers Locomotive Works, at Paterson, N. J.

He was quick to perceive the great field for development which lay in the branch of the profession which he subsequently adopted, and his name will be prominently associated for all time with those members of the profession who have done so much in the past 40 or 50 years to study, develop, and perfect the science and art of Structural Engineering.

His first work in this line was with the Michigan Bridge and Construction Company, of Detroit, Mich., with which Company he accepted a position as Assistant Engineer in 1871. In 1873, he took charge of the Engineer's Office of the Erie Railroad Company in New York City, under the late Octave Chanute, Past-President, Am. Soc. C. E., Chief Engineer. In this position, one of Mr. Schneider's duties was to check the strain sheets and plans submitted by bridge companies. Up to this time this had not been the general practice, the railroads depending mainly on the bridge companies for the correctness of the designs. He also organized a thoroughly trained inspection force. At this time experienced inspectors could hardly be obtained, and the work was generally accepted without inspection. Mr. Schneider selected a number of young men from the office and shops, who showed practical ability, and gave them instructions which enabled them to inspect the work in the shops, both as to quality and progress.

Bridge work up to this time had usually been let on a competitive lump-sum basis. Mr. Schneider soon found that this method was unsatisfactory, and the Railroad Company's officials decided to make their own plans; and it was Mr. Schneider's duty to prepare them. The bridge requirements for 1874 were designed in this way, and the work was let on a pound-price basis, and this, in all probability,

* Memoir prepared by the following Committee: Paul L. Wolfel, Richard Khuen, Jr., and O. E. Hovey, Members, Am. Soc. C. E.

was the first instance where bridge work was let by the pound, a procedure which Mr. Schneider always favored.

In 1876 and the early part of 1877, he was engaged by the Board of Engineers (sometimes called the Steinway Commission), appointed by the Long Island Bridge Company, to prepare and pass on plans for a railroad and highway bridge across the East River, to connect Long Island with New York City. It was the intention to locate this bridge between 76th and 77th Streets, this site having been selected as the narrowest point in the East River. The plan required a 734-ft. span across the west channel and a 618-ft. span across the east channel. The project, however, was not carried out at this time, on account of the financial condition of the Company.

From May, 1877, to July, 1878, Mr. Schneider was associated with the Delaware Bridge Company, of New York, as a Designer. Charles Macdonald, Past-President, Am. Soc. C. E., was President of this Company. During this period the Delaware Bridge Company made an arrangement with the Edge Moor Iron Company, of Wilmington, Del., to manufacture the ironwork of the bridges built by the former company, and Mr. Schneider was stationed at Edge Moor where he had charge of the design and construction of various bridges, including the Rockville Bridge over the Susquehanna River, near Harrisburg, Pa., on the Pennsylvania Railroad, the Cohoes Bridge, on the Delaware and Hudson Railroad, and several smaller ones.

On August 1st, 1878, he opened his own office in New York City, specializing in the design of bridge and structural work. One of the first pieces of work handled by him was a number of Howe truss spans for the Canadian Pacific Railway, the most important of which was for the Stony Creek Viaduct. This bridge carried traffic for the Canadian Pacific Railway until 1893, when it was replaced by a large arch span. Mr. Schneider's connection with the Canadian Pacific Railway Company dates from this time. For this Company he acted as Consulting Engineer, in numerous cases, almost to the end of his life.

One of the most important structures for this railroad was the 527-ft. cantilever bridge over the Fraser River, built by Mr. Schneider in 1887. This was one of the first cantilever bridges built in America, and it carried traffic until 1910, when it was taken down to make room for a structure of greater capacity; but it was re-erected across a chasm known as Niagara Ravine, on the line of the Esquimault and Nanaimo Railway (operated by the Canadian Pacific Railway), near Victoria, B. C. Both these structures were considerably ahead of their time, and have not been improved on by the most recent practice, except in a very few features.

In more recent years, Mr. Schneider passed on the design of the Lethbridge Viaduct, for the Canadian Pacific Railway, and also quite a number of smaller structures, including some Scherzer and Strauss bascule bridges. He was also called in consultation by this Railway Company for the reconstruction of the St. Lawrence River Bridge, near Montreal, Que., where the rebuilding of the superstructure and the remodeling of the piers for a wider and heavier bridge involved numerous difficult and interesting problems.

Numerous other structures were handled by Mr. Schneider in his New York office, the most important of which was the cantilever bridge over the Niagara River for the Grand Trunk Railroad. He wrote a highly interesting paper* about this structure for the Society, for which he was awarded the Rowland Prize in 1886.

He also designed all the interior steel framework and the anchorage for the Statue of Liberty in New York Harbor during this time.

When, in 1886, competitive designs were requested for a crossing over the Harlem River, by the Commission appointed by the City of New York, Mr. Schneider was awarded first prize for his plans. His friends have always regretted that his very beautiful and correctly conceived design had not been built in place of the structure which is now known as the Washington Bridge.

From 1879 to 1883, Mr. Schneider was associated with the late George S. Morison, Past-President, Am. Soc. C. E., on a number of important structures, such as the Plattsmouth, Bismarck, and Blair Bridges across the Missouri River, and the Snake River Bridge, at Ainsworth, Wash.

In 1886, Mr. Schneider was a member of a board of consulting engineers appointed by the New York District Railway Company. His associates on this board were: William P. Trowbridge, Charles C. Martin, Julius W. Adams, John T. Fanning, Alfred P. Boller, Gen. Quincy A. Gilmore, Henry Morton, and Charles F. Chandler. This Company proposed to build an underground railway under Broadway, with a junction at 14th Street, one line passing up Madison Avenue and the other continuing north under Broadway.

In May, 1886, Mr. Schneider entered into an agreement with the A. and P. Roberts Company, of Philadelphia, Pa., owners of the Pencoyd Iron Works, to establish a Bridge and Construction Department in connection with its works, and, subsequently, was appointed Chief Engineer. Under his direction this Bridge and Construction Department developed into the largest and most progressive establishment of its kind in the United States, and gained an international reputation.

* "The Cantilever Bridge at Niagara Falls", *Transactions*, Am. Soc. C. E., Vol. XIV, p. 499.

Some of the most important structures built by the Pencoyd Iron Works under his supervision are the Delaware River Bridge, for the Pennsylvania Railroad, near Philadelphia; the Niagara Falls and Clifton Arch Bridge, over the Niagara River, near the Falls; the Pennsylvania Railroad Company's old trainshed, in Jersey City; the Chesapeake and Ohio Bridge down the James River, in Richmond; and innumerable smaller structures in the United States, Mexico, and Japan.

In 1893, the Long Island Railroad Company took an interest in the project for a bridge over the East River at Blackwell's Island, previously referred to, with a view of establishing a terminal in New York City. Work was actually commenced, and, in 1894, competitive designs for the bridge were invited. The bridge was to accommodate four railroad tracks, with approaches and a terminal station west of Second Avenue, New York City. The plans adopted by the Long Island and New York Bridge Company were the designs made by Mr. Schneider, for the Pencoyd Iron Works, to which Company the contract for the entire steel superstructure of the bridge and approaches was awarded in March, 1895. Considerable work had been done on this bridge on piers and foundations, complete detailed drawings of the superstructure were made, and a portion of the material was rolled, when, on account of the death of Austin Corbin, President of the Long Island Railroad, the Company again became disorganized.

The American Bridge Company was formed in 1900, and, on May 21st, Mr. Schneider was elected Vice-President in charge of Engineering. He held this office until May 16th, 1901, when it was abolished and, on the same day, he was elected a Director and Vice-President of the American Bridge Company of New York, also in charge of Engineering. These offices he held until April 20th, 1903, when he became Consulting Engineer of the Company, which position he held during the remainder of his life. His associates in the American Bridge Company held him in the highest esteem, and felt that his influence for good in the Company was very potent, particularly in the early days of the organization. The first President of the Company expresses his estimate of him as follows:

"Mr. Schneider without question stood at the very head of his Profession and, in addition, I believe, never had an enemy in his entire career. I say this from intimate personal contact extending over a period of fifteen or twenty years. He had the confidence, not only of his fellow engineers, but of the consumer as well. His position at the head of the Engineering Department of the American Bridge Company gave to the organization a solid foundation among its competitors and the confidence of its customers. To the internal working of the Company, he was a great advantage at the start, as naturally many conflicting interests had to be considered, and I believe the entire staff was always willing to accept his decisions without friction."

During this time, Mr. Schneider continued his connection with the Canadian Pacific Railway as Consulting Engineer, and was also appointed Consulting Engineer for the Baltimore and Ohio Railroad Company.

In 1903, he was commissioned, in conjunction with Theodore Cooper, M. Am. Soc. C. E., by the Imperial Government Railways of Japan, to prepare a large set of standard plans for bridges for the Japanese Railways. These plans are still in use, and large tonnages have been shipped from the United States to Japan in conformity with them.

After the collapse of the Quebec Bridge, he was commissioned, by the Canadian Government, to make a report on the causes of the failure, which report he finished, in 1908, in the most thorough and exhaustive manner. In 1911, he was appointed by the Canadian Government as a member of the Board of Engineers for the rebuilding of the Quebec Bridge, which position he held until his death.

Mr. Schneider frequently contributed to technical papers. Besides his article on the cantilever bridge at Niagara Falls in 1886, previously mentioned, for which he received the Rowland Prize, in 1905 he received the Norman Medal from the Society for his paper on "The Structural Design of Buildings",* and, again, in 1908 the Norman Medal for his paper on "Movable Bridges."† These, together with his "Standard Specifications for Railway and Highway Bridges" and the volume of "Standard Details" which he compiled for the American Bridge Company, known to every structural engineer, form his chief contributions to technical literature.

When Mr. Schneider wrote his first railroad specifications for the Pencoyd Iron Works, he put his impact theory into practical use for the first time. This method of calculation has been adopted by the Government of India and by a large number of American railways, and has also been incorporated in the "Manual" of the American Railway Engineering Association and in its "General Specifications for Railway Bridges."

Mr. Schneider was a member of the American Railway Engineering Association, American Society for Testing Materials, the Verein Deutscher Ingenieure, in Germany, and the Engineers' Club of New York.

Mr. Schneider was dearly beloved by his many friends on account of his sterling character and his kindly disposition. He was always willing and ready to assist brother engineers with advice, giving to them freely from his rich fund of knowledge, and large indeed is the number of engineers to-day in responsible positions, who owe their training and their positions to him. He was most democratic in his ways and of a lovable disposition, and gained, in the highest degree,

* *Transactions*, Am. Soc. C. E., Vol. LIV, p. 371.

† *Transactions*, Am. Soc. C. E., Vol. LX, p. 258.

the respect of everybody who came in contact with him. He always stood for good work, good designs, and good details, and the Engineering Profession is greatly indebted to him for the present high standard that has been obtained in bridge and structural work. His was a most useful life, well lived, an example and an inspiration to the Profession, that will remain in the memory of all who had the privilege of knowing him.

He was married on January 8th, 1880, to Catherine Clyde, daughter of John J. and Ruth H. (Luther) Winters, of New York City. It was a great blow to him when he lost his only son in early boyhood. He is survived by his widow and his daughter, Helen, the wife of John Phillips Badenhausen, M. Am. Soc. C. E.

Mr. Schneider was elected a Member of the American Society of Civil Engineers on February 6th, 1884. He served as a Director in 1887 and from 1898 to 1900, and was elected Vice-President in 1902 and President in 1905. He was Chairman of the Library Committee in 1903 and Chairman of the Committee on Concrete and Reinforced Concrete from 1904 to 1911.

WILLIAM DOUGLAS PICKETT, Hon. M. Am. Soc. C. E.*

DIED MARCH 5TH, 1917.

William Douglas Pickett was the son of George Blackwell Pickett and Courtney (Heron) Pickett, members of prominent Virginia families. After their marriage, the Picketts had settled in the Tennessee River Valley, near Huntsville, Ala., where William Douglas Pickett was born on October 2d, 1827. After the death of his father, the family moved to Kentucky where the boy received his preliminary education, first in Richmond and then in Lexington. Having chosen Civil Engineering as his life work, he prepared himself for it at Transylvania University, from which he was graduated in 1845.

Immediately after his graduation, Mr. Pickett went to Texas, where he was engaged as Engineer in establishing the disputed lines of a large landed estate. While thus employed, war with Mexico was declared, and Mr. Pickett enlisted with the Texas Rangers. In 1848 when the war was ended, he returned to Kentucky and resumed the practice of his Profession. He was engaged on the reconstruction of the strap-railroad from Lexington to Frankfort, the first railroad built west of the Alleghany Mountains. He also served as Assistant Engineer on the railroad constructed from Lexington to Danville, and, later, as Chief Engineer, he located and built railroads in Tennessee, Georgia, Alabama, and Arkansas.

At the outbreak of the Civil War in 1861, Mr. Pickett was employed as Chief Engineer on the construction of the Memphis and Ohio Railroad in Tennessee. He resigned immediately and enlisted in the Confederate Army. He raised a company of civil engineers and received a commission as its Captain from the Governor of Tennessee. Shortly afterward, he was transferred to the staff of Major-General Pillow and designed and constructed a number of water batteries along the Mississippi River. While serving as Engineer Aide to General Leonidas Polk, Captain Pickett was requested by General Polk to accompany him to Columbus, Ky., to witness the test of a new gun. On the first discharge, the gun burst, killing a number of the men who served it, the concussion hurling both General Polk and Captain Pickett through the air. Although he escaped with his life, Captain Pickett's hearing was permanently impaired by the shock. He was afterward transferred to the staff of Major-General Hardee as Military Engineer, and took part in the battles of Shiloh, Perryville, Murfreesboro, and Missionary Ridge, as well as the engagements around Atlanta.

When General Hardee was transferred to a new Department, with headquarters at Charleston, S. C., Captain Pickett went with him,

* Memoir prepared by the Secretary from information on file at the Society Headquarters.

and was actively engaged in the siege of Savannah, the evacuation of Charleston and Fort Sumter, and the march of Hardee's Army to a junction with that of General Johnston, and, on the surrender of the latter, in April, 1865, Captain Pickett was paroled as Colonel and Inspector General of Hardee's Division.

After a venture in cotton planting, unsuccessful on account of floods in Yazoo Pass, Colonel Pickett resumed the practice of his profession, and, as Chief Engineer, rebuilt the bridges and reconstructed the roadbed for the Memphis and Ohio Railroad Company, now a part of the Louisville and Nashville System.

On April 27th, 1870, Colonel Pickett was married to Miss Theodosia Curd, of Lexington, Ky., who died a year later, leaving a son who survived her only four years.

After the deaths of his wife and little son, Colonel Pickett went West in 1873, and after several years spent in prospecting and fighting Indians over the territory now embraced in the Yellowstone Reservation, he established himself on a cattle ranch in the Big Horn Country, in Wyoming, and became widely known as a successful hunter of the grizzly bear. When this region became populated and prosperous, Colonel Pickett represented his district for several terms in the State Legislature of Wyoming, first as Representative and afterward as Senator.

In 1904, he returned to Kentucky and purchased a home in Lexington, where he lived until his death on March 5th, 1917, in his ninetieth year. Knowing the hopelessness of his case, he met his end like a soldier, calmly and unflinchingly, his last days being disturbed only by the thought of leaving his invalid brother, Major George Blackwell Pickett, who survives him.

A life-long friend, Judge George B. Kinkead, of Lexington, Ky., writes of Colonel Pickett as follows:

"Few knew him intimately, but to many the tall, alert figure of this handsome old gentleman, democratic in his feelings, but displaying the patrician in every lineament, was familiar. In every walk of life, his most striking and distinguishing characteristic was a prompt recognition of every duty imposed, and an unfaltering courage to discharge it. One who lived close to him for years, but a few days since remarked to the writer, 'He was the best neighbor I ever knew.'"

Colonel Pickett was a member of the Boone and Crockett Club, and the Confederate Veterans' Association. He was also a member of Christ Church Cathedral, from which his funeral was held on March 7th, 1917.

He was the oldest living member of the American Society of Civil Engineers and although not present at the meeting of November 5th, 1852, when the Society was organized, he was elected a Member on July 6th, 1853, and therefore might be classed as a Charter Member.

Throughout his long membership, he took a lively interest in the welfare and prosperity of the Society and on all occasions proudly displayed his badge. In 1908, Colonel Pickett, then in his eighty-first year, contributed a discussion on the paper by H. M. Chittenden, M. Am. Soc. C. E., entitled "Forests and Reservoirs in Their Relation to Stream Flow with Particular Reference to Navigable Rivers"*, and also a paper entitled "The Floods of the Mississippi Delta: Their Causes, and Suggestions as to Their Control".† In 1911, Colonel Pickett attended the Annual Convention of the Society, which was held that year at Chattanooga, Tenn.

Colonel Pickett was elected a Member of the American Society of Civil Engineers on July 6th, 1853, and an Honorary Member on April 1st, 1914.

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* *Transactions*, Am. Soc. C. E., Vol. LXII, p. 423.

† *Transactions*, Am. Soc. C. E., Vol. LXIII, p. 53

JOHN FERRIS ALDEN, M. Am. Soc. C. E.*

Died February 27th, 1917.

John Ferris Alden, the son of Sidney and Harriet Webster Alden, and a direct descendant in the eighth generation of John Alden, of Plymouth, Mass., was born at Cohoes, N. Y., on March 19th, 1852. He was educated in private schools in Albany, N. Y., and at Rensselaer Polytechnic Institute, from which he was graduated with honors, in June, 1872, with the degree of Civil Engineer.

Mr. Alden did his first engineering work as Assistant Engineer on the construction of the railroad bridge to carry the tracks of the New York Central Railroad over the Hudson River at Albany, N. Y., under Mr. Charles Hilton as Chief Engineer.

In January, 1875, Mr. Alden accepted a position as Assistant Engineer at the Leighton Bridge and Iron Works at Rochester, N. Y. In 1877, he was made Chief Engineer and, in 1878, became a member of the firm. On July 1st, 1881, with the late Moritz Lassig, M. Am. Soc. C. E., as partner, he leased the Leighton Bridge and Iron Works and continued the business in Rochester under the name of Alden and Lassig and that in Chicago, Ill., under the name of Lassig and Alden.

In January, 1886, the partnership was dissolved and Mr. Alden purchased the business interests of the firm in Rochester, Mr. Lassig retaining those in Chicago. Mr. Alden then re-organized his business under the name of the Rochester Bridge and Iron Works of which he continued as sole owner until 1901 when he sold out to the American Bridge Company. He retained an interest in the Company, however, until his death.

During his active connection with the Rochester Bridge and Iron Works, Mr. Alden built many steel and iron bridges, especially for railroads, throughout the United States and Canada. Among these may be mentioned the Elevated Railroad in New York City; bridges for the Delaware and Hudson Railroad, the Chicago, Milwaukee and St. Paul Railroad, and the Buffalo, Rochester and Pittsburgh Railroad; the bridge over the Columbia River at Pasco, Wash.; the viaducts at Los Angeles, Cal.; the Upper Suspension Bridge at Niagara Falls; and the Driving Park Avenue Bridge at Rochester, N. Y. He also constructed the tower and elevator in the House of Parliament, at Ottawa, Ont., Canada, as well as many buildings in New York, Chicago, and other cities, and furnished most of the iron and steel work for the buildings of the Columbian Exposition, at Chicago, Ill.

* Memoir prepared by the Secretary from information on file at the Society Headquarters.

Mr. Alden was married, in 1885, to Miss Mary E. Bogue, of Brooklyn, N. Y., who, with two sons and three daughters, survives him.

At the time of his death which occurred at his home at Rochester, N. Y., on February 27th, 1917, after a brief illness, Mr. Alden was President of the Locke Insulator Manufacturing Company and a Director of the Genesee Valley Trust Company. He was also a member of the Rensselaer Society of Civil Engineers, the Rochester Chamber of Commerce, the Alden Kindred of America, the Sons of the Mayflower, the Genesee Valley Club, and the Rochester Club. He was Vice-President of the Board of Trustees of the Friendly Home, and was actively interested in the erection of its new home. He was also a Vestryman and prominent member of Christ (Protestant Episcopal) Church.

Mr. Alden was elected a Member of the American Society of Civil Engineers on July 6th, 1887.

THOMAS APPLETON, M. Am. Soc. C. E.*

DIED AUGUST 3D, 1916.

Thomas Appleton, the son of Edward and Frances Anne Atkinson Appleton, was born at Reading, Mass., on October 13th, 1846. He was of Colonial ancestry, his father's family having settled in Ipswich, Mass., in 1635. His mother was a descendant of Simon Bradstreet, an early Colonial Governor of Massachusetts and New Hampshire, and Anne Dudley.

Mr. Appleton's father, Edward Appleton, was a Civil Engineer, having been graduated from Harvard College in the Class of 1835, and a member of the first Board of Railroad Commissioners of Massachusetts. In 1862, after completing his studies in the public schools of Reading, Thomas Appleton entered the employ of the Boston, Hartford and Erie Railroad as Rodman, continuing with this Company until he enlisted, for the Civil War, in Company E, Eighth Massachusetts Volunteer Infantry, from which he was honorably discharged in November, 1864.

In 1865, Mr. Appleton entered Rensselaer Polytechnic Institute, from which he was graduated with the Class of 1868 with the degree of Civil Engineer. Among his classmates at Rensselaer were M. T. Endicott, Past-President, Am. Soc. C. E., and the late Virgil G. Bogue, L. L. Buck, and O. F. Nichols, Members, Am. Soc. C. E.

After his graduation, Mr. Appleton was appointed Resident Engineer with the Dexter and Newport Railroad Company, on construction work in Maine, and, in 1869, he was made Division Engineer on the Somerset Railroad. In 1870, he was appointed Engineer in charge of the reconstruction of the Troy and Greenfield Railroad (now a part of the Fitchburg Division of the Boston and Maine System) east of the Hoosac Tunnel, in Massachusetts. Recently, the old steel work on this line was replaced to accommodate the heavier loads and train movements of these days, but the abutments of the two bridges across the Deerfield River, constructed under Mr. Appleton's direction, were not disturbed or strengthened.

On the completion of this work, in 1871, Mr. Appleton was appointed Chief Engineer of the Essex Branch and Marginal Freight Railroads in Massachusetts, and, in 1872, he was made Principal Assistant Engineer of the Boston and Maine Railroad.

From 1873 to 1879, he was engaged on surveys for several railroads, and acted as Inspector of Bridges for the City of Boston and also as Traveling Agent for the Niagara Bridge Works, of Buffalo, N. Y.

In 1880, Mr. Appleton was made Superintendent of Bridges and Buildings for the Galveston, Harrisburg and San Antonio Railway

* Memoir prepared by the Secretary from information on file at the Society Headquarters.

(now a part of the Southern Pacific System in Texas) and, in 1881, he was engaged in making location surveys in New York and Connecticut for the New York, Boston, Albany and Schenectady Railroad. He left the latter work to become Chief Engineer of the Kansas and Eastern Construction Company and, in this capacity, built the Leavenworth, Topeka and Southwestern Railroad. In 1883, he was employed as Assistant Engineer on the Topeka, Salina and Western Railroad.

In 1884, Mr. Appleton was appointed City Engineer of East Saginaw, Mich., but preferring railroad work, he resigned to accept the position of Principal Assistant Engineer on the Chicago, Milwaukee and St. Paul Railroad, which he held from 1885 to 1889. He then went to the Union Pacific Railroad where he served as Engineer of Buildings until the latter part of 1890. The late Mr. Bogue, one of his classmates at college, was Chief Engineer of the Union Pacific Railroad at that time, and on hearing of Mr. Appleton's death, he wrote of him as follows: "I always had a great affection for him and always found him a sweet-tempered, kindly-disposed man, who did his duty."

In 1891, Mr. Appleton opened an office in Chicago, Ill., for the private practice of engineering, which he continued until 1895, when he was appointed Engineer of Buildings and Water Supply with the Great Northern Railroad. He resigned this position in 1898 to accept that of Chief Engineer of the Copper Range Railroad.

In 1900, Mr. Appleton entered the Government service as Assistant to the Superintendent of Construction of the United States Life Saving Service, in which capacity he had charge of the construction of many of the life-saving stations along the Atlantic Coast and the Great Lakes.

In 1902, at his own request, Mr. Appleton was transferred from the Life Saving Service to the office of the Supervising Architect, Treasury Department, as Superintendent of Construction, United States Public Buildings, which position he held until his death at Gardiner, Me., on August 3d, 1916. His first work while holding this office was on the construction of the Post Office Building at Creston, Iowa. This was followed by work on public buildings at Holyoke and Northampton, Mass., Wheeling, W. Va., Evanston, Wyo., East St. Louis, Alton, Centralia, and Greenville, Ill., Chelsea, Mass., the new Custom House at Boston, Mass., New Bedford, Mass., and Gardiner, Me. After the earthquake and fire in San Francisco, he was ordered there to assist in the work of temporary repairs to protect Government buildings. The deep and difficult foundations for the new Custom House at Boston, Mass., were constructed under his supervision, but at his request and on account of advancing years, he was relieved from that detail and transferred to New Bedford in 1913.

Mr. Appleton was married to Mary Louise Burnham, of Essex, Mass., on October 1st, 1871, and is survived by his widow, one brother, four sisters, and two stepsons.

He was the founder and first President of the Society of Constructors of Federal Buildings, which was organized, principally through his efforts, in 1910, with the idea of promoting closer relations between the Office and Field forces of the Supervising Architect's Office, and thereby raising the efficiency of both. The success of this organization, which has grown from a few members to include not only the entire Field force of the Office, but also a considerable number of the Office force, is due to Mr. Appleton's never-failing interest and activity in its affairs. He attended all its Annual Conventions and was a generous contributor to the pages of its *Journal,* his last paper, on the history and origin of the Society, having been published in May, 1916.

In 1874, Mr. Appleton joined the Boston Society of Civil Engineers and continued his membership in that organization for many years after going West. He was a charter member of the American Engineering and Maintenance of Way Association (now the American Railway Engineering Association), and while in Chicago, he joined the Western Society of Engineers and served as its Secretary for a year or more. He was also a member of the American Society of Mechanical Engineers from 1893 to 1910, a member of Kilpatrick Post No. 71, G. A. R., of Holyoke, Mass., and of the Zeta Psi Fraternity.

The following extract from a letter from the Acting Supervising Architect, Mr. James A. Wetmore, to the Secretary of the Society of Constructors of Federal Buildings, expresses the high estimate in which Mr. Appleton was held by all who knew him:

"* * * In every relation with him, official and personal, I was impressed with his devotion to duty, his ideals of rectitude and honor, his kindly disposition, and his consideration for others. Endowed with a pleasing personality, possessed of the requisite technical qualifications, gifted with tact and a disposition to find a way to overcome difficulties, it was natural that he should be highly regarded by his associates and co-workers in the Office and in the Field. I feel that in his death, the public service has lost a faithful servant, your Society a devoted adherent, and all of us a sincere friend."

Mr. Appleton's immediate superior, Mr. George O. Von Nerta, Technical Officer of the Supervising Architect's Office, writes:

"* * * Where he stood out apart from the rest was in his ability to conceive and carry into execution an improvement of far-reaching effect."

Mr. Appleton was elected a Member of the American Society of Civil Engineers on April 4th, 1883.

WILLIAM HARRY ARNOLD, M. Am. Soc. C. E.*

DIED AUGUST 13TH, 1917.

William Harry Arnold was born at Smithfield, Pa., on September 22d, 1864. From 1880 to 1883, he was a student at the Mt. Pleasant Classical and Scientific Institute, at Mt. Pleasant, Pa., and from January, 1883, to August, 1885, he served as an Apprentice in the Erecting Shops of the Pennsylvania Railroad at Pittsburgh, Pa.

In September, 1885, Mr. Arnold entered Rensselaer Polytechnic Institute, at Troy, N. Y., from which he was graduated in June, 1890, with the degree of C. E., having lost one year (1886), on account of sickness.

In June, 1890, he entered the employ of Nier and Hartford, of Chattanooga, Tenn., being engaged on general engineering work. He remained in this position and with Hartford and Hebert, until June, 1891, when he was engaged by G. W. G. Ferris and Company, of Pittsburgh, Pa., on general inspection work.

From April to September, 1895, Mr. Arnold served with the Barber Asphalt Company, of Buffalo, N. Y., and then came to New York City, as Office Manager for R. W. Hildreth and Company. He also had charge of the field work for this Company in the vicinity of New York.

From August, 1896, to April, 1899, Mr. Arnold was engaged with various officers of the Corps of Engineers, U. S. A., generally around New York City, first as Draftsman, and then as Inspector of harbor improvements. This work included the Patchogue Jetty improvement, the dredging of the South Amboy, N. J., Channel, the dredging of the Bay Ridge and Red Hook Channels, New York Harbor, as well as work in connection with battery emplacements at Fort Wadsworth, N. Y., and Fort Taylor, at Key West, Fla.

In April, 1899, Mr. Arnold entered the employ of the W. H. Beard Dredging Company, of New York City, as Superintendent and Engineer, in complete charge of the dredging fleet and the reconstruction of dredges and tugs, retaining this position until July, 1903, when he became Superintendent of the fleet of the Hughes Brothers and Bangs Dredging Company, of New York City.

From July to October, 1904, Mr. Arnold was employed by J. G. White and Company, of New York City, on harbor work in the Philippine Islands.

In March, 1905, he was engaged by the Bush Terminal Company, of New York City, first as Superintendent of Floating Equipment,

* Memoir prepared by the Secretary from information furnished by E. B. Ashby, Cons. Engr., Lehigh Valley Railroad Company.

including the design and construction of tugs, car floats, dredges, etc., until June, 1906, when he was made Chief Engineer of the Company.

In 1907, Mr. Arnold resigned his position with the Bush Terminal Company to enter into private practice as a member of the firm of Arnold and Andrew, Consulting Engineers, but, in 1909, the partnership was dissolved, and Mr. Arnold became General Manager of the Bay State Dredging Company, of Boston, Mass.

From 1912 until January, 1914, he was engaged in general engineering work. On the latter date he was appointed Terminal Engineer of the Lehigh Valley Railroad Company, in New York City, which position he retained until his death, on August 13th, 1917. In this capacity, Mr. Arnold had charge of the construction of the new ore dock at Constable Hook, N. J., as well as the piers, pier sheds, and dredging in New York Harbor and at Perth Amboy, N. J.

Mr. Arnold was elected a Member of the American Society of Civil Engineers on May 1st, 1907.

EDWARD MANNING BIGELOW, M. Am. Soc. C. E.*

DIED DECEMBER 6TH, 1916.

Edward Manning Bigelow was born in Pittsburgh, Pa., on November 6th, 1850, being one of five children born to Edward M. and Mary Steel Bigelow. He received his early education in the public schools and after graduation entered the Western University of Pennsylvania (now the University of Pittsburgh) where he studied civil engineering.

Upon leaving college he entered the service of the City of Pittsburgh in a subordinate position on the staff of the City Engineer, which position he held until 1874 when he was promoted to engineer in charge of certain street construction, working under a Commission which was authorized by special legislative enactment. Subsequently, the laws authorizing this work were declared unconstitutional, and Mr. Bigelow left the service of the City for a short time. He was re-employed by the City and served as Assistant Engineer in charge of surveys and the location of streets from 1876 to 1878, and in charge of construction from 1878 to 1882.

On January 9th, 1882, he was unanimously elected City Engineer by Council, which position he held until 1885, and from 1885 to 1888 he was City Engineer and Commissioner of Highways. At this time the City Charter was changed, the Department of Public Works being created, and Mr. Bigelow was then appointed Director of the Department of Public Works, which office he held for two full terms of three years each.

On June 1st, 1911, Governor John K. Tener of Pennsylvania appointed Mr. Bigelow to the office of State Commissioner of Highways, which office he held until April, 1915. Mr. Bigelow was honored by a re-appointment to the office of Director of the Department of Public Works of Pittsburgh just prior to his death.

Mr. Bigelow's entire life was devoted to municipal improvements. He was an engineer of broad vision and foresight; his plans for improvements were always comprehensive, and provided well for the future. Under his supervision many miles of Pittsburgh's streets and sewers were laid out and constructed, including the boulevards. His greatest achievement, and the one in which he took the most pride, was the development of Highland and Schenley Parks. It was also under his administration that work on the water purification plant was started.

Mr. Bigelow was an honest, tireless, and conscientious worker, and more than ordinary credit is due him from the people of Pittsburgh

* Memoir prepared by N. S. Sprague, M. Am. Soc. C. E.

when it is considered that many of the improvements which he inaugurated and carried through to successful completion were consummated in the face of much criticism and abuse. The appreciation of his services in the development of the parks was manifest by the erection of a bronze statue placed at the entrance of Schenley Park, the cost of which was defrayed by public subscription. He was known as the "Father of the Parks".

The same characteristic plan on a large and comprehensive scale was displayed by Mr. Bigelow during his tenure of office as Commissioner of Highways of the State of Pennsylvania. Under his direction plans were made for the improvement of a comprehensive system of highways traversing the State in anticipation of securing funds for their improvement. Many of the State highways were improved, and much credit is due to him for the creditable work performed with the limited funds at his disposal.

Even during periods when Mr. Bigelow was not connected with the government of Pittsburgh, his interest in city affairs never ceased; and his genius for planning and energy in advocating improvements for the benefit of the city were always manifest.

Aside from Mr. Bigelow's professional career, he was a man of affairs and took an active interest in financial, commercial, and social activities. He was a Trustee of the Carnegie Institute of Technology from the time of its founding, and took an active part in the development of this institution. He was President of the Liberty National Bank, the Liberty Savings Bank, and the Homewood Cemetery. He was also a Trustee of Andrew Carnegie's benefactions, the Carnegie Library and Institute, Carnegie Hero Fund Commission, the Institute of Technology, and also of the Schenley Memorial Commission. He was a member of, and also took an active part in, the East Liberty Presbyterian Church, and his time and talents were given freely in the raising of funds and for other purposes connected with this church.

Upon Mr. Bigelow's death, the Council of the City of Pittsburgh paid due respect by calling a special meeting of that body for the purpose of honoring the memory of the late Director of the Department of Public Works, and the following resolution was passed, which perhaps is the best expression of the high regard and esteem in which his services to the City of Pittsburgh are held by this community:

"*Whereas,* Almighty God, in His infinite wisdom, has seen fit to remove from our midst one of Pittsburgh's most distinguished citizens; and

"*Whereas,* Edward Manning Bigelow was a man of stainless integrity, of superb courage, and of great intellectual force; a man who always walked in the highway of right; and in disaster stood erect; a man to whom defeat was but a spur to further effort; a man who

believed in the loyalty to man, in the sovereignty of the citizen, and in the matchless greatness of this grand City; a man who gave freely of his talents and genius to the up-building of the City of Pittsburgh, who served the City for many years with a self-sacrificing devotion, and whose master hand is indelibly impressed upon every section of the City; and

Whereas, As he lived he died, with that tenacity of purpose which characterized his life; while yet in love with life and raptured with the world he bravely answered the call of the Master of the Universe and passed to his eternal home beyond the utmost reach of human harm or help. He has left with us his wealth of thought and deed—the memory of a brave, imperious, honest man, who bowed alone to death; therefore be it

Resolved, that a copy of these resolutions be spread upon the Minutes of Council of Pittsburgh, and that a copy be sent to his bereaved widow."

In addition to the foregoing resolution, many tributes of respect were paid to Mr. Bigelow's ability as a citizen and public officer by men prominently identified with the financial, commercial, and professional affairs of the City.

Mr. Bigelow was elected a Member of the American Society of Civil Engineers on December 4th, 1889.

DANIEL WHEELER BOWMAN, M. Am. Soc. C. E.*

Died March 14th, 1917.

Daniel Wheeler Bowman, the son of Quaker parents, Francis Bowman and Elizabeth (Hammond) Bowman, was born in New Bedford, Mass., on October 2d, 1844. In 1857 his parents purchased a farm at Ledyard, Cayuga County, N. Y., and moved there. Mr. Bowman spent much time on this farm, lending to his life the enthusiasm and interest which marked him throughout his later years. There he acquired and developed his great love for Nature and growing things; and this clung to him until his death.

He received his early education at Oakwood Seminary, Union Springs, N. Y., from which he was graduated in 1865. He returned and took a post-graduate course at the Seminary, on finishing which he was given charge of a Friends Academy, in Ohio. In 1868, Mr. Bowman went to Kansas on some survey work, returning the same year to enter Cornell University where he took the Civil Engineering Course. His life at Cornell was characteristic of the man: simple, honest, and earnest. With an unusually alert mind and great application, he stood very high in all his classes, and by his native kindness and interest in his fellows and in all that pertained to the general welfare, he endeared himself to his classmates and all who knew him. He was graduated in 1872 in the first "through" class in Civil Engineering. In 1883, Cornell University conferred on him the degree of C. E.

Immediately after his graduation, Mr. Bowman was offered a position as Instrumentman on some survey work for the New York Central Railroad Company in New York State, which he accepted, and in which position he continued for two years, resigning to become associated as Engineering Secretary to the late Capt. James B. Eads, M. Am. Soc. C. E., who at that time was in charge of important jetty work at the Southwest Pass of the Mississippi River. While on this work, Mr. Bowman was directly under the late G. W. R. Bayley, M. Am. Soc. C. E., and was sent to New York City and Washington, D. C., on special missions in connection with his work. He also had charge of the construction of the dredge, *G. W. R. Bayley,* in Pittsburgh, Pa., in 1877, for use on the jetty work. At that time it was the largest dredge built, and was an engineering triumph. While on this work Mr. Bowman married Ida Portia, the daughter of Mr. Bayley. Two sons and a daughter by this marriage survive him.

* Memoir prepared by N. R. McLure, M. Am. Soc. C. E., with the assistance of Seymour P. Thomas, Esq.

After this work was finished, Mr. Bowman made some very extensive computations for a proposed bridge across the East River at Blackwell's Island, New York City, for Capt. Eads, in connection with the competition then asked for by those contemplating this structure. His ability as a Designing Engineer on this work was so marked that he was sent by Capt. Eads to Phœnixville, Pa., in connection with some work that was then being done for him by Clarke, Reeves and Company (now The Phœnix Bridge Company). At that time the late Adolphus Bonzano, M. Am. Soc. C. E., was Chief Engineer of this concern, and was so attracted by the quick perception, ability to grasp and solve difficult situations, and general engineering ability of young Bowman, that he arranged for him to remain at Phœnixville as his Assistant, in charge of estimating and designing. While in this position Mr. Bowman designed the first Kinzua Viaduct, for the New York, Lake Erie and Western Railroad Company, in McKean County, Pa., which, at that time, was one of the longest and highest railroad viaducts in the world. He designed the Albany and Greenbush Bridge across the Hudson River, at Albany, N. Y., a double-deck structure carrying railroad and roadway, with approach spans and a rim-bearing draw-span. He also designed the difficult reverse curve on the New York Elevated Railway work, at 110th Street, Boulevard Crossing. This was built with Phœnix columns and latticed girders, and, at the time, was considered quite a complicated structure. Another notable bridge designed by Mr. Bowman at this period was that over Rondout Creek, on the West Shore Railroad.

In 1885, Mr. Bowman resigned his position as Designing Engineer of The Phœnix Bridge Company and went South, doing some bridge inspection work for the Southern Railway, and, in 1888, he was made Bridge Engineer for the Central of Georgia Railway. While in this position he had charge of the rebuilding of many of the old bridges on the line. In 1891, he was made Assistant Engineer of The Boston Bridge Works, at Boston, Mass., which position he filled until 1894, when he returned to Phœnixville, Pa., as Assistant Engineer for The Phœnix Iron Company. In 1896, he went to Chicago, Ill., with several associates, and organized the Universal Construction Company, of which he became Chief Engineer. In 1897, he again returned to Phœnixville as Assistant Engineer for The Phœnix Iron Company, under H. H. Quimby, M. Am. Soc. C. E., then Chief Engineer. On the resignation of Mr. Quimby from this Company in November, 1900, Mr. Bowman succeeded him as Chief Engineer, in charge of the estimating, computing, designing, detailing, and erection of the structural steel fabricated by The Phœnix Iron Company. He continued as Chief Engineer until February, 1914, when he was made Consulting Engineer, which position he held at the time of his death.

In 1899 he was married to M. Virginia Vanderslice, of Phœnixville, Pa. The years of Mr. Bowman's best service were devoted to structural steel, particularly as applied to bridge and building work, and there are few localities of size in the eastern section of the United States which have not, however unconsciously, felt the influence of his ability through structures erected in their midst. These were improved in construction by his ever watchful and able supervision of the design.

Mr. Bowman was broad and liberal in his views of life, a progressive in more than one sense of the word, charitable in his nature, and a friend and counsellor to those who sought his help and advice. Many a young engineer has been started on the right path through his careful and painstaking efforts in his behalf. He was a great lover of Nature, and spent many days in his later life, in the open among his flowers and fruit trees. He had an intense and loyal affection for Cornell, his Alma Mater, and seldom missed returning at Commencement time to renew his youth with the younger Alumni. Among a host of friends and acquaintances, rarely was there a man who will be so sadly missed.

Mr. Bowman was elected a Member of the American Society of Civil Engineers, on December 3d, 1912.

CHARLES ADOLPHUS CALDWELL, M. Am. Soc. C. E.*

DIED AUGUST 31ST, 1916.

Charles Adolphus Caldwell, the son of Oscar Adolphus Caldwell and Carrie B. Caldwell, was born in Macon, Bibb County, Ga., on October 18th, 1866.

He began his engineering studies in 1882 at Vanderbilt University where he remained two years. In 1884, he entered Rensselaer Polytechnic Institute, Troy, N. Y., and was graduated in 1888 with the degree of C. E.

Shortly afterward Mr. Caldwell began his professional career under F. T. Dabney, Chief Engineer of the Central of Georgia Railroad, on the extension of this road from Savannah to Americus.

In 1890, he was made Assistant Engineer on the St. Augustine and North Beach Railroad, in Florida, from which position he resigned to return to Macon, to accept an appointment in the office of the City Engineer. In 1894 he was associated with Capt. J. W. Wilcox, Assistant Engineer on the construction of the Fort Valley Water-Works, and as Principal Assistant to the City Engineer on the construction of 32 miles of sanitary sewers in Macon, Ga.

In 1898, he was the Principal Assistant Engineer, in charge of construction work, for the Macon Gas Light and Water Company, during the construction of a 5 000 000-gal. filter plant and a new pumping station.

In 1899, Mr. Caldwell designed and supervised the sanitary sewer system for the City of Valdosta, Ga., and, in the same year, as Chief Engineer, designed and constructed a water-power development for 1 000 h.p., at Juliet, Ga., on the Ocmulgee River. Here he also built a 3 000-spindle cotton mill, and a grist mill with a daily capacity of 8 000 bushels for the Juliet Milling Company.

From 1899 to 1907, he was engaged in the general practice of his profession, with offices at Macon, Ga. During this time, he had charge of a large amount of general engineering work, including a number of highway bridges, a sanitary sewer system for Forsyth, Ga., and was associated with J. N. Hazlehurst, M. Am. Soc. C. E., in making the appraisal of the existing water-works property of the Macon Gas Light and Water Company, and designing a proposed municipal water-works system of 10 000 000 gal. daily capacity. In 1910, he and Mr. Hazlehurst represented the City of Macon before the Board of Arbitration which fixed the value of this property at $699 000, when the plant was purchased by the City.

* Memoir prepared by G. R. Solomon, M. Am. Soc. C. E.

In 1911, Mr. Caldwell prepared reports and plans on a 10 000-h.p. hydro-electric development on the Ocmulgee River, near Juliet, and was also engaged, as Chief Engineer, on the remodeling and extension of the water-works system of the City of Macon.

In addition to his engineering work, Mr. Caldwell was interested in several manufacturing concerns as an officer and decorator, and his opinion was always sought and valued because of his fair-mindedness. He was universally liked and trusted, and his death is a decided loss to the Profession.

He was unmarried, and is survived by one sister, Mrs. John B. Birch, of Macon, Ga.

Mr. Caldwell was elected a Member of the American Society of Civil Engineers, on September 5th, 1911.

EDWARD CANFIELD, M. Am. Soc. C. E.*

DIED AUGUST 18TH, 1916.

Edward Canfield, son of Caleb Augustus and Sarah Withington Canfield, was born at Geneva, N. Y., on May 27th, 1848. He was graduated from Hobart College in 1869, receiving the degree of A. B., and, in 1874, the degree of A. M.

In 1869, Mr. Canfield began his professional career in the Engineering Corps of the Erie and Geneva Valley Railroad. From 1870 to 1872, he was Division Engineer of the Syracuse Northern Railroad; from 1872 to 1873, Division Engineer of the Pennsylvania Petroleum Railroad; from 1873 to 1874, Assistant Engineer, Atlantic and Great Western Railway; from 1875 to 1878, Division Engineer of the Syracuse, Geneva and Corning Railroad; from 1879 to 1881, Assistant Engineer of the New York, Lake Erie and Western Railroad; and from 1881 to 1882, Roadmaster of the Rochester Division of the New York, Lake Erie and Western Railroad. While in the latter position, he organized and carried out the work of changing the entire Rochester Division from broad to standard gauge in one day.

On November 9th, 1882, Mr. Canfield entered the service of the New York, Ontario and Western Railway Company, in which he continued to the close of his life, having been first appointed Superintendent of Construction of the line from Middletown to Cornwall, N. Y., and from there to Weehawken, N. J., the latter portion being now a part of the West Shore Railroad, which is leased to and operated by the New York Central Railroad Company. In 1884, on the completion of the construction work, he was appointed Superintendent of the Southern Division.

On December 1st, 1887, he was appointed Chief Engineer in charge of new construction, maintenance of way, and bridges and buildings. As Chief Engineer, he had charge of locating and constructing the branch from Cadosia, N. Y., to Scranton, Pa., which gave the New York, Ontario and Western Railway Company entrance into the anthracite coal regions of Pennsylvania. In 1895, he was appointed General Superintendent, having charge of all operation except traffic, which office he held at the time of his death.

Mr. Canfield served for some years on the Board of Directors of the Orange County Trust Company, was, at the time of his death, a Trustee of the Middletown Savings Bank, and, at one time, a member of the Board of Education of Middletown, N. Y.

* Memoir prepared by J. H. Nuelle, Esq.

The Board of Directors of the New York, Ontario and Western Railway Company adopted the following minute at a meeting held September 12th, 1916:

"This Board records its deep sense of loss in the death of Edward Canfield, late General Superintendent, which occurred at his home in Middletown, New York, on August 18, 1916.

"Mr. Canfield entered the service in 1882 in the Engineering Department, later becoming Chief Engineer, and in 1895 General Superintendent.

"In all his long service he displayed intense loyalty to the Company and its interests; to his skill as an engineer is due in large part the continued progressive advance in the physical condition of the property, while through his good judgment and careful management a high degree of efficiency in operation has been attained.

"His fine character, marked ability, untiring industry, absolute devotion to duty, and unfailing justice in his relations with all in the service, gained for him the profound respect and regard of all who knew him.

"No better example could be found in railroad service than the record of his life work."

He was married on September 17th, 1874, to Jane Hastings, daughter of Major David H. Hastings, of the United States Army, who, with their sons, David H. Canfield, Captain Edward Canfield, U. S. A., and Richard W. Canfield, survives him.

Mr. Canfield was a member of the Sigma Phi Fraternity, and was elected a Member of the American Society of Civil Engineers on December 3d, 1879.

ANDREW CHASE CUNNINGHAM, M. Am. Soc. C. E.*

DIED JANUARY 13TH, 1917.

Andrew Chase Cunningham, the son of Thomas and Celeste (Chase) Cunningham, was born at Mohawk, N. Y., on February 15th, 1858.

He was appointed to the United States Naval Academy from the Twenty-first District of New York State, on June 9th, 1874, and was graduated from that institution as a Midshipman on June 10th, 1879. He served on the U. S. S. *Shenandoah* and the U. S. S. *Saratoga*, and on February 1st, 1883, he resigned from the service, meanwhile having been promoted to the rank of Ensign. Mr. Cunningham had decided by this time that a career in the Navy was not what he desired, and, having determined to become a Civil Engineer, he entered Rensselaer Polytechnic Institute, at Troy, N. Y., from which he was graduated in 1885 with the degree of Civil Engineer.

From October, 1885, to April, 1886, Mr. Cunningham occupied the position of Topographer on preliminary and location surveys of the Lincoln Branch of the Missouri-Pacific Railway, and from May, 1886, to September, 1887, he served as Draftsman with the Massillon Bridge Company, Massillon, Ohio. From September, 1887, to November, 1890, he was in charge of the inspection of iron and steel in Pittsburgh, Pa., and vicinity. This work consisted in the acceptance or rejection of material for such structures as the high bridge across the Mississippi River, at St. Paul, Minn., the Ohio Connection Bridge across the river below Pittsburgh, the New York Elevated Railway, numerous large buildings in Chicago, Ill., and bridges on the Lake Shore and Michigan Southern Railway, the Pennsylvania System, the Louisville and Nashville, and other railroads.

From November, 1890, to May, 1892, Mr. Cunningham was Chief Inspector for Carnegie, Phipps and Company, of Pittsburgh, Pa., now the Carnegie Steel Company. In this position he had charge of the testing and inspection of steel materials, together with special investigation and special supervision of material for several structures, such as the Memphis Bridge, the Sixth Street Bridge across the Allegheny River at Pittsburgh, and others.

In May, 1892, Mr. Cunningham associated himself with Charles F. Stowell, M. Am. Soc. C. E., at Albany, N. Y., under the firm name of Stowell and Cunningham, the principal engineering business of the company being in connection with the design, inspection, and testing of steel bridges and steel materials. This work included materials and bridges for the New York Central and Hudson River Railroad, and the Central Vermont Railroad, as well as materials for the Cities of

* Memoir prepared by F. T. Chambers, M. Am. Soc. C. E.

Albany, N. Y., Waterbury, Conn., for the State Engineer of New York, and the U. S. Treasury Department. In 1898, while he was a member of this firm, the Spanish War broke out, and, as a former officer of the Navy, he obeyed his country's call, volunteering for such duty as he might be assigned to by the Navy Department. At that time the Navy was by no means as large as it is now, and had almost no auxiliary ships for the purpose of furnishing supplies to the fleet. Mr. Cunningham, therefore, found himself an Ensign, on May 14th, 1898, aboard the Collier *Abarenda,* a ship purchased for that use. While this ship was moored to the wharf at the New York Navy Yard, the late L. L. Buck, M. Am. Soc. C. E., visited the Navy Yard for consultation work, in connection with one of the dry docks then under repair. Some one had informed him that Mr. Cunningham was serving as an Ensign aboard the collier, and Mr. Buck expressed the determination to visit him before he left the yard, adding at the same time that he considered Mr. Cunningham one of the foremost steel experts of the country, and that it seemed to him a waste of valuable services to have placed him as an Ensign aboard a supply ship. Mr. Cunningham's nature was such that he would never have sought to be transferred from this position, but Mr. Buck felt strongly on the subject, and made it his business to inform the Navy Department of his views, the consequence being that on May 21st, 1898, Mr. Cunningham was transferred to the Bureau of Ordnance, and was immediately assigned to the Washington Navy Yard, which is the Naval gun factory. Mr. Buck and Rear-Admiral Mordecai T. Endicott, U. S. N. (*Retired*), Past-President. Am. Soc. C. E., were classmates at Rensselaer Polytechnic Institute, and it was not long before Admiral Endicott, then Chief of the Bureau of Yards and Docks, learned that an accomplished Civil Engineer was available for naval duty, and, consequently, Mr. Cunningham was transferred to the Bureau of Yards and Docks on June 27th, 1898. The Civil Engineer Corps of the Navy was then rapidly expanding and in need of officers, and Mr. Cunningham served first on a board to examine candidates for the position of Civil Engineer in the Navy; before the successful candidates were appointed, however, he himself, on September 29th, 1898, was given a permanent commission. On October 5th of that year, he reported to the Bureau of Yards and Docks, where he served until November 6th, 1901, being then detached and ordered to the Naval Station, New Orleans, La. This was a new station just being established, and Mr. Cunningham was the first Civil Engineer officer, thus being given the opportunity to lay out the engineering works from the start. On April 3d, 1903, he was detached from the New Orleans Station and ordered to the Naval Academy, where he served until June 9th, 1905, and from there he was again ordered to the Bureau of Yards and Docks. On March 17th,

1906, he was commissioned with the rank of Lieutenant in the Corps of Civil Engineers.

Mr. Cunningham continued to serve in the Bureau of Yards and Docks as Principal Assistant to Admiral Endicott until April 6th, 1907, when he was ordered to the Navy Yard, Norfolk, Va., as Senior Civil Engineer Officer, and on November 18th, 1909, he was commissioned with the rank of Lieutenant Commander. On February 20th, 1910, he was detached from the Norfolk Navy Yard, and ordered to the Navy Department, Washington, D. C., as Inspector of Public Works for the entire Navy. This duty required great tact and diplomacy, and it was on this account that Mr. Cunningham was selected for the work, which necessitated his maintaining headquarters in Washington, and visiting the various Navy Yards, keeping the Department informed as to the status of the various public works, and co-ordinating the ideas of the Yards with those of the Department.

On July 10th, 1913, Mr. Cunningham left headquarters at Washington to assume the duties of Public Works Officer of the Navy Yard, Portsmouth, N. H. Shortly afterward he had a severe nervous breakdown, and on November 17th, 1913, was ordered to sick leave, and did not return to actual duty until June 16th, 1914. He never fully recovered from this illness, although he performed lighter duties practically up to the time of his death, his principal assignments after this being at the Naval Training Station, Great Lakes, Ill., and the Naval Radio Station, Point Isabel, Tex., at both of which places he erected large steel towers for radio-telegraphy.

While on duty in the Bureau of Yards and Docks of the Navy Department, and previous to his detail as Civil Engineer Officer at New Orleans, Mr. Cunningham had supervised the construction of the 16 000-ton steel floating dock for the New Orleans Station. He became much interested in docks of this type, and, at a later date, obtained letters patent on a floating dock of his own invention. He was, indeed, of an inventive turn of mind and secured patents on several of his ideas, one of the best known in the Navy being that for a coal-tar paint.

Mr. Cunningham was affectionately known as "Andy" by his friends and associates, and was universally liked. His genial nature, combined with his diplomatic spirit, caused him to be much in demand on boards of officers for the adjustment of disputes or for changes in contracts.

While at the Naval Academy, he was the champion fencer of his time, and he maintained his interest in this sport up to the time of his severe illness. While in Washington he was a member of the Washington Fencers Club, and when he was at the Navy Yards he stirred the younger men to a revival of the fencing game. He was looked on by the entire Navy as an authority on this subject, and was also consulted by the Army at one time, in connection with the modification

of the Army saber. As a fencer, he was also interested in singlestick, and was the author of a book entitled "The Cane as a Weapon." Fencing was a considerable feature of his recreation; he was also very fond of writing, and contributed various articles to the press, among them being several on naval matters published by the Naval Institute.

Mr. Cunningham was married, at Middleville, N. Y., on June 18th. 1879, to Miss Jessie E. Thomas. He is survived by his widow and two sons: John Howard Cunningham of Grand-Mère, Canada, and George Thomas Cunningham, of Washington, D. C.

Mr. Cunningham was elected an Associate Member of the American Society of Civil Engineers on September 2d, 1891, and a Member on October 3d, 1894.

DAVID WEST CUNNINGHAM, M. Am. Soc. C. E.*

DIED MAY 11TH, 1916.

David West Cunningham, the son of Andrew and Abigail Leonard (West) Cunningham, was born on December 24th, 1829, in Boston, Mass. He was educated at Chauncy Hall School and the Lawrence Scientific School of Harvard University.

In 1848, Mr. Cunningham received an appointment in the Engineering Corps of the Boston Water-works, then building the Cochituate Aqueduct. In 1849, he was on railroad work on the Manchester and Lawrence Railroad and the Sullivan Railroad of the Connecticut River, from which he was transferred to the Ogdensburg and Lake Champlain Railroad. He was stationed first at Moline and afterward at Lawrenceville on the Moira Sub-division, and rode on the first locomotive through to Ogdensburg, which was driven by Mr. Charles L. Schlatter, then Chief Engineer. He was then employed on railroad work in Canada. The eastern half of the Ogdensburg and Lake Champlain Railroad at that time ran through primitive forest, and while at this work, he contracted inflammatory rheumatism which affected his eyes, and he was compelled to give up engineering work for the time being. In 1851, he joined a friend who was a merchant at Cienfuegos, Cuba, and sailed from Boston for Spain early in June, arriving at Cadiz in 24 days.

After 6 months' travel in Europe, Mr. Cunningham entered the Lawrence Scientific School at Harvard, for a course in Civil Engineering, and was graduated as a Civil Engineer.

Mr. Cunningham spent six years in Chili, on railroad and government work, but left to be married. He never returned to that country, on account of his wife's preference for the United States. After leaving Chili, he operated a plantation in Texas.

After the Civil War Mr. Cunningham was engaged in the construction of the Charleston, Mass., Water-works, the Lowell Water-works, and the sewerage systems of Lowell, Mass., and Stillwater, Minn. He was Chief Assistant Engineer for the Boston Water-works. He built the Tarkio Valley (Missouri) Railroad, and later was Consulting Engineer for the Minneapolis Water-works.

Mr. Cunningham then worked a large wheat farm in North Dakota until 1894, at which time he removed to California.

He was married in 1859 to Mary B. S. Fuller, of Boston, who died in 1869. In 1873, he was married to Caroline Smith Thomas, who died at Los Angeles, Cal., in 1910. In 1913, he was married to Minnie A. Holderbaum, at Santa Ana, Cal.

* Memoir prepared by Frank H. Olmsted, M. Am. Soc. C. E.

Mr. Cunningham was a man of high scientific attainments, and naturally a builder. He was favorably known wherever his duties carried him. He was genial in disposition, loved his friends, and had few, if any, enemies. He could without doubt have reached a high position had he remained in the Profession and not digressed at one or two periods in his career to other kinds of work. He was a remarkably well read man, and spoke and wrote fine English. He was also an excellent Spanish scholar, and a valued member of the various scientific bodies to which he belonged.

Mr. Cunningham was a member of the Architects and Engineers' Association of Southern California. He was elected a Member of the American Society of Civil Engineers on May 7th, 1873.

EDMUND HAZEN DRURY, M. Am. Soc. C. E.*

DIED JANUARY 31ST, 1917.

Edmund Hazen Drury was born at St. John, New Brunswick, on July 31st, 1859. He was graduated with honor from the Royal Military College, Kingston, Ont., in June, 1881, and immediately joined the Engineering Staff of the Canadian Pacific Railway Company, continuing thereon until 1888, first as Rodman and finally as Division Engineer.

From 1889 to 1890 Mr. Drury served as Assistant Engineer on railway surveys for the Dominion Government in New Brunswick, and from 1890 to 1893, he was Division Engineer and Acting Chief Engineer on the McLeod Branch of the Calgary, Edmonton, and McLeod Railway.

From 1893 to 1906, he held the following positions: Chief Engineer and Manager of Construction for the contractors of the Quebec Central Railway, 1893-95; Division Engineer on the Canadian Northern Railway, 1895-97; Division Engineer on the Bangor and Aroostook Railway, 1897-98; Assistant Chief Engineer on the Canadian Northern Railway, 1898-1900; Assistant Chief Engineer, Cuba Company Railway, Cuba, 1900-02; Assistant Chief Engineer, Algoma Central Railway, 1902-03; Division Engineer, Crow's Nest Pass Branch, Canadian Pacific Railway Company, 1903-05; and Division Engineer, Canadian Northern Manitoba South Eastern Railway, 1905-06.

Mr. Drury then went to Mexico where he was employed as Auditing Engineer and Assistant Chief Engineer in charge for the Mexican Light and Power Company, Mexico, in Necaxa and the City of Mexico, from 1906 to 1908.

In the autumn of 1908, he was appointed, by the Department of Railways and Canals of Canada, Engineer in charge of exploration and the reconnaissance survey of alternative routes to Port Churchill and Port Nelson on Hudson Bay. His report, which was an able and comprehensive one, was concluded and handed in toward the end of 1909.

In 1910, he was appointed Chief Engineer of the Quebec and Sherbrooke Railway, which position he held until 1911, when he was engaged to prepare an estimate and report for a proposed railway from Edmonton, Alberta, to Bella Coola, on the Pacific Coast of British Columbia, *via* the Pine River Pass, Rocky Mountains, for the Edmonton, Dunvegan, and Pine Pass Railway Company.

Mr. Drury then went to South America as Chief Engineer and Manager of Construction of the Chili Longitudinal Railway, Chili, from 1911 to 1913. It was while conducting this work, through the

* Memoir prepared by E. J. Walsh, Esq.

arid nitrate district of Chili, that his health became impaired. Returning to Canada, in the autumn of 1913, he entered into a general consulting engineering partnership (in the firm of Walsh and Drury, Consulting Engineers), with an office at Ottawa, Ont.

From the latter part of 1914 to the time of his sudden death on January 31st, 1917, Major Drury had served as Acting Assistant Director General of Engineer Services, for the Department of Militia and Defence, at Ottawa.

Major Drury stood in the forefront of the profession as a Railway Engineer. He was a man of high character and unswerving integrity, and neither in private nor public affairs would he deviate from the conscientious discharge of life's duties. Kindly and courteous to a degree, he will be greatly missed by those of his confreres and others who were intimate with him.

Major Drury was a Member of the Canadian Society of Civil Engineers, and was elected a Member of the American Society of Civil Engineers on October 4th, 1905.

AUGUSTUS JAY DuBOIS, M. Am. Soc. C. E.*

DIED OCTOBER 19TH, 1915.

Augustus Jay DuBois, the son of Henry Augustus DuBois and Catherine Helena (Jay) DuBois, who had six other children, was born at Newton Falls, Ohio, on April 25th, 1849. His father, who was of French Huguenot descent, received the degree of M.D. from Columbia College in 1830 and spent most of his life in the practice of medicine. His mother was a granddaughter of Chief Justice John Jay, who was also of French Huguenot descent.

Mr. DuBois prepared for college at the Hopkins Grammar School in New Haven, Conn., and then took the course in Civil Engineering at the Sheffield Scientific School of Yale University, from which he was graduated in 1869. Continuing there in advanced studies, he secured the degree of C. E. in 1870 and that of Ph.D. in 1873. He then spent 18 months at the Royal Mining Academy in Freiberg, Saxony, followed by a few months of surveying work in California and Connecticut.

During 1871-75 he made a special study of the then new science of Graphic Statics, the results of which were published in 1875, in two volumes, under the title "Elements of Graphical Statics and Their Application to Framed Structures." This was the first comprehensive work on the subject which appeared in the United States, and it was re-issued in revised editions in 1877, 1879, and 1883.

In 1875, Mr. DuBois was appointed Professor of Civil and Mechanical Engineering in Lehigh University, from which he was called, in 1877, to the chair of Mechanical Engineering at the Sheffield Scientific School, and, in 1884, he was appointed Professor of Civil Engineering there, a position which he filled until his death.

During his forty years of service as a teacher of Engineering, Professor DuBois was active in enriching the theory of the subject. He translated from the fourth edition of Weisbach's "Mechanics of Engineering", the sections "Hydraulics and Hydraulic Motors" (published in 1877), and "Theory of the Steam Engine" (1877); also Weyrauch's "Calculation of Iron and Steel Constructions" (1877), and Roentgen's "Principles of Thermodynamics" (1879). These books were issued at a time when literature on these subjects was scanty in the United States, and they were used extensively in engineering schools and by engineers, each of them passing through several editions. In 1883 appeared the first edition of his "Stresses in Framed Structures," a book in quarto form, which gave methods of computing stresses by

* Memoir prepared by Mansfield Merriman, M. Am. Soc. C. E.

both analytic and graphic processes and also contained fifteen folding plates of designs and working drawings of bridges. This book was widely used as a text in engineering schools, and it passed through twelve editions, each being revised and improved. The manuscript for an enlarged edition was completed shortly before his death.

In 1894 and 1895 there appeared three octavo volumes by Professor DuBois, entitled "Elementary Principles of Mechanics", one treating of kinematics, another of statics, and a third of kinetics. In 1901, these volumes were re-issued in revised quarto form as the first volume of "Mechanics of Engineering", the second volume being the twelfth edition of his "Stresses in Framed Structures."

Professor DuBois had contributed papers on roof trusses, retaining walls, and the steam engine to the technical journals; he had also taken part in discussions before the Society on flexure of beams, and had contributed papers on "The Weights of Bridges"* and on "The Strength of Columns".†

During 1889-94 he prepared and delivered six lectures entitled "Science and Faith", "Science and Immortality", "Science and Miracle", etc., which were published in the *Century Magazine* and other periodicals. These lectures were marked by originality of thought and beauty of style, and by the purpose to establish moral truths on the fundamental principles of mechanics; one of the last products of his pen was to summarize the conclusions of these lectures in an article in the *Yale Review* for July, 1913.

Professor DuBois was a hard worker, a clear and logical writer, and his books greatly advanced the interests of sound education in theoretical and applied mechanics. As a teacher, he was most successful, and especially was he insistent that students should acquire a thorough knowledge of fundamental principles. His successor, Professor John C. Tracy, in an obituary notice in the *Yale Alumni Weekly*, wrote as follows:

"A sympathetic interest, a ready wit, and a friendly unconventional manner won his students from the start. He was a clear and original thinker, and a keen but sympathetic critic. Breadth of culture and an unusual power of expression made him a brilliant and inspiring conversationalist. Underneath a quiet and undemonstrative exterior, there was a man chivalrous, sympathetic, always thoughtful of others, loyal, and wholly lovable. Only a few of his closest friends knew how, in his own quiet unostentatious way, he went about doing good, and to them he seemed an almost perfect type of Christian gentleman."

Professor DuBois rarely attended engineering meetings, seeming to feel somewhat awkward outside of the circle of his friends and

* *Transactions*, Am. Soc. C. E., Vol. XVI (1887), p. 191; Vol. XVIII (1888), p. 170.

† *Transactions*, Am. Soc. C. E., Vol. XXVIII (1892), p. 69.

students. In his college days he was a good chess player and a member of the Book and Snake Fraternity, but he took little interest in other social activities. He made six trips to Europe, for rest and relaxation during summer vacations, but he never had a Sabbatical year in whole or in part during his forty years of service as a teacher.

He was married, on June 23d, 1883, to Miss Adeline Blakesley, daughter of Arthur Blakesley, of New Haven, Conn. They had no children, and she survived him only seven months.

Professor DuBois was a member of the American Institute of Mining Engineers, the American Society of Mechanical Engineers, and the Society of Naval Architects and Marine Engineers, as well as several scientific academies and associations.

Augustus Jay DuBois was elected a Junior of the American Society of Civil Engineers on July 7th, 1875, and a Member on October 5th, 1892.

JOHN WALDO ELLIS, M. Am. Soc. C. E.*

DIED OCTOBER 30TH, 1916.

John Waldo Ellis, son of John and Ame Almira Fisher Ellis, was born in Woonsocket, R. I., on September 7th, 1845. He was one of four children, of which three were boys, all noted civil engineers.

Mr. Ellis received his early education at New Hampton Institute, New Hampton, N. H., and at the age of 19 entered the employ of the Boston, Hartford and Erie Railroad, then building from Waterbury, Conn., to Fishkill, N. Y. His advancement was rapid, and he soon became a Division Engineer on the Troy and Greenfield Railroad before the completion of the Hoosac Tunnel. From this road he went to the Norwich and Worcester Railroad then building, and, in 1869, came to Woonsocket, R. I., as Chief Engineer of the Providence and Worcester Railroad, at the same time opening an office for private practice in that place, which he maintained up to within a few years of his death.

Mr. Ellis held the position of Chief Engineer of the Providence and Worcester Railroad, up to the time that road was absorbed by the New York, Providence and Boston Railroad in 1888. Under his direction the road was double-tracked, many branch lines were constructed, the Wilkes-Barre Coal Pier and connection was constructed at Providence, and many bridges, stations, and terminals were rebuilt. During this same period Mr. Ellis' private practice in Woonsocket was at its height, and many prominent engineers of the present day received their first experience in the old Main Street office. The design and direction of the construction of Nourse Mill, of the Social Manufacturing Company, the Alice Mill, of the Woonsocket Rubber Company, and numerous other industrial plants and enlargements in Northern Rhode Island, were a part of the activities of this office.

From 1890 to the time of his death, Mr. Ellis was connected prominently with the various engineering problems of the East. He was Engineer for the Old Colony Railroad Company in the building of the Providence Passenger Terminal, and Engineer Inspector of the Boston and Providence Railroad, from the time of its lease to the Old Colony Railroad, until his death. His connection with various grade crossing matters in Massachusetts and Rhode Island included nearly every important problem that has come up. He was one of the Commissioners for the abolishment of the grade crossings in Lowell, Athol, and Orange, Mass., and was employed as engineering expert by the Cities of Lynn, Worcester, Cambridge, Fall River, Taunton, Haverhill, Readville, and a large number of small towns.

* Memoir prepared by Lester Waldo Tucker, M. Am. Soc. C. E.

As a Water-Works Expert, Mr. Ellis was among the foremost in New England, serving as one of the Commissioners in the valuation of the Newburyport and Gloucester Water-Works when these were taken over by the City. He was also a member of the Commission in the diversion claims against the City of Pittsfield and the claim of the Nassau Paper Company against the Metropolitan Water Board.

As a Town and City Engineer, Mr. Ellis was especially active, serving as Town Engineer of Woonsocket from 1870 to the time it became a city in 1888. He also served as Engineer for the Town of Blackstone, and for other surrounding towns, up to the time of the closing out of his private practice.

As a Hydraulic Engineer, Mr. Ellis was very active, and the Blackstone and other rivers in Massachusetts and Rhode Island have many a dam constructed under his direction. The most prominent of these are the Lonsdale, Ashton, and Wilkinsonville Dams on the Blackstone; the Slatersville Reservoir Dam and Middle Dam on the Branch River, and the Georgiaville Dam on the Woonasquatucket, in Rhode Island.

On March 1st, 1901, Mr. Ellis was elected President of the Providence Gas Company, and took up the active management of that Corporation, holding the position of President and General Manager to the time of his death. Under his direction, this Company became one of the most efficient of the gas companies operating in the East.

Notwithstanding his many engineering engagements and business connections, Mr. Ellis found time to be a most efficient Director and Manager in other fields. He was a member of the Board of Directors of the Industrial Trust Company, of Providence, and Chairman of the Board of the Woonsocket Branch of that Company. He was a Director in the Woonsocket Rubber Company, and many other Corporations. He was also a Trustee of the Woonsocket Institution for Savings, from 1876 to 1908, and a Trustee of the Woonsocket Hospital from its founding, in 1890, to the time of his death. Although a prominent member of many clubs and social organizations, Mr. Ellis had no connection with any fraternal or secret orders. He was a member of the Boston Society of Civil Engineers, serving as President of that Society in 1905. He was also a member of the New England Water Works Association.

His principal diversion in his leisure was that of driving. From the time when he established his home in Woonsocket his stable always contained at least one good blooded trotting horse, and when the roads were good or the sleighing at its best, Mr. Ellis was to be seen among the fastest of those on the speedways. He was a member of the Woonsocket Driving Club, the Roger Williams Driving Club, of Providence, and the Metropolitan Driving Club, of Boston, and it is

interesting to note that only three weeks before his death he drove on the track of the latter Club.

Mr. Ellis was a prominent figure in the political field of Woonsocket for many years. He served as Alderman from his Ward from 1895 to 1897 and was President of the Board during the last two years of this service. In 1904, he was elected State Senator from his city and served on many important Committees.

He was a member of the Board of Trustees of the First Universalist Church for many years.

Mr. Ellis was a man of such marked ability in any of the fields into which he entered that he was recognized as an authority on an unusual range of engineering problems, a public man and a statesman of great ability, and a business man with keen foresight and tremendous energy. The scope of his talents was wide, and indicated a breadth of mentality seldom found in one man.

Mr. Ellis was married on May 23d, 1870, to Mary F. Howe, who, with one son and two daughters, survives him.

He was elected a Member of the American Society of Civil Engineers on July 3d, 1895, and served as a Director from 1904 to 1906, inclusive.

THEODORE NEWEL ELY, M. Am. Soc. C. E.*

DIED OCTOBER 28TH, 1916.

Theodore Newel Ely was born in Watertown, N. Y., on June 23d, 1846. When he retired from the office of Chief of Motive Power of the Pennsylvania Railroad System on July 1st, 1911, he had lived through and participated in the most active period of railroad development that the world has ever seen, having begun his railroad career in 1868.

Mr. Ely was graduated from the Jefferson County (N. Y.) Institute in 1863, and from the Rensselaer Polytechnic Institute, of Troy, N. Y., with the degree of Civil Engineer, in the class of 1866.

After his graduation he had an opportunity to enter the United States Navy, the Navy Department having offered to accept the Institute's examinations as the equivalent of those at Annapolis and grant a full cadetship after a year of sea experience; another offer was that of a Professorship at Rensselaer. He preferred, however, active professional work.

After a brief experience in the office of the City Regulator (City Engineer) of Pittsburgh, Pa., he went to the Fort Pitt Foundry, where he became associated with Gen. Thomas J. Rodman, the inventor of the well-known Rodman gun. The Government specifications for the metal for the guns were very exacting, and Mr. Ely here obtained his first experience in the investigation on a commercial scale of the qualities of metals. After the completion of this work, he was engaged for a short time in coal mining.

In 1868 he entered the Engineer Corps of the Pittsburgh, Fort Wayne and Chicago Railroad, at Pittsburgh. In July of the same year he accepted an offer of the position of Assistant Engineer on the Philadelphia and Erie Railroad, a part of the Pennsylvania Railroad System. In 1869, he was made Superintendent of the Middle Division, with headquarters at Renovo, Pa., and, in 1870, Assistant General Superintendent of the road. Some of the features of Mr. Ely's work during this period were re-alignments of the road, the rebuilding of practically every bridge on the Philadelphia and Erie, and the building of the docks and terminals at Erie Harbor, to take care of the road's prospective combined lake and rail traffic. During this period, also, the first steps were taken in the development of the Pennsylvania Railroad standards of materials and designs, and, in connection with Mr. Frank Thomson, who was Superintendent of the Eastern Division of the Philadelphia and Erie, he designed and made detailed drawings—the first issued at any point on the road—for a standard section of

* Memoir prepared by George L. Fowler, Esq., New York City.

track. A sample mile of this track established its superiority over the previous construction, and, with the exception of the weight of the rail, its general features are still followed. He also conducted, in connection with Mr. William Robinson, an elaborate series of experiments with electric signals, which were fully developed later.

His duties as Assistant General Superintendent of the Philadelphia and Erie included the supervision of the Motive Power Department, of which he took direct charge in 1873, on his appointment as Superintendent of Motive Power.

In 1874, Mr. Ely was made Superintendent of Motive Power of the Pennsylvania Railroad Division, and, in 1882, General Superintendent of Motive Power of all the Lines East of Pittsburgh. His office in both these positions was at Altoona, Pa. Railroad conditions during Mr. Ely's administration at Altoona did not call for a radical change in the types of locomotives or cars, and their development, therefore, was largely in increasing the efficiency of the types then in general use by making important modifications and perfecting their details. He left his imprint on the locomotives of the country by the simple and clean-cut character of the designs developed at Altoona, which eliminated the useless and out-of-place ornamentation which was then a feature of locomotive construction. The same is true with regard to passenger equipment cars, for the improvement of which Mr. Ely's good taste was largely responsible. Among the practices introduced during this period for increasing the efficiency and economy of operations, the following may be briefly mentioned.

At the time of Mr. Ely's appointment as Superintendent of Motive Power, the purchase of materials used was not controlled by specifications of any kind, the brand or reputation of the seller being depended on as a measure of quality. Recognizing that competitive purchasing required as its foundation a clear description of the article to be purchased, Mr. Ely obtained in 1875 permission to establish a Department of Tests, both physical and chemical, to develop the necessary specifications for the materials to be bought, and to examine the materials received to insure that they met the requirements of the specifications. Mr. Ely wisely selected young and ambitious men as the heads of the two branches of the Test Department, one being John W. Cloud and the other the late Charles B. Dudley, M. Am. Soc. C. E., and his firm support of his subordinates was largely responsible for the success of the new plan, as purchasing on the basis of specifications and tests alone naturally aroused strong opposition from many manufacturers, particularly those with established brands. In some cases, however, the manufacturers themselves were the strongest advocates of the general use of the specifications. The new Department speedily developed great value in the investigation of many of the technical questions which were constantly arising in railroad work, and it has made

notable contributions in the line of scientific investigations covering a wide range of subjects.

Realizing the increasing responsibilities of the Mechanical Department, Mr. Ely encouraged the entrance into the service of young men of technical education, who were given a course of practical training in the shops under the same rules and regulations as the regular apprentices, their subsequent advancement depending on their development of the necessary qualifications.

The piecework system, which is now used on the whole Pennsylvania System, was introduced during the early part of Mr. Ely's administration. Another feature was the extensive use of committees for the study of important questions.

Mr. Ely's encouragement and the assistance given by his organization were large factors in the introduction and successful development of the automatic car coupler, the air brake, and other features of modern railroad practice.

An important undertaking was the establishment at Altoona, in 1889, of the Juniata Shops for building locomotives. These shops have been extended from time to time since to keep pace with the increasing size of modern locomotives, and still build the greater number of locomotives required by the Company.

Another notable practice established under Mr. Ely's direction was the pooling of the freight cars of all the lines of the Pennsylvania Railroad System, so far as repairs were concerned.

Incidents of this period were the active part which Mr. Ely took in the rebuilding of the road at Johnstown, after the flood in 1889; his work as a member of the Commission on Safe and Vault Construction, appointed by Act of Congress in 1890, for the purpose of improving the vault facilities of the Treasury Department; the exhibits of the Pennsylvania Railroad Company at the various International Expositions—the Centennial, Philadelphia, 1876; Paris, 1889; Columbian, Chicago, 1893; as well as the later ones, Louisiana Purchase, St. Louis, 1904; and Jamestown, 1907—which were prepared and carried out under his direction.

In 1893, Mr. Ely was appointed Chief of Motive Power of the Pennsylvania Railroad Lines East and West of Pittsburgh, with office at Philadelphia. Although in this position his duties were supervisory, he kept in close touch with the work of development of locomotives and passenger and freight cars to meet the changing conditions, both as to traffic and as to materials for car construction. He gave close study to and took an active part in the consideration of the questions of policy involved in the increases in the capacities of locomotives and cars, especially in freight service; the substitution of steel for wood in the construction of both freight and passenger cars; and the establishment of the general principles of the designs.

Other activities during this period were his work as Chairman of the Rail Committee of the System, charged with the improvement of the rail used; as a member of various committees of the American Railway Association; as a member of the Permanent Commission of the International Railway Congress; as President of the Eastern Railroad Association, dealing with patent matters; as a Trustee of the Philadelphia Commercial Museum, which is devoted to the development of the foreign trade of the United States; and as a member of the Pennsylvania State Commission on the Chestnut Tree Blight. He was also a Director, in the interest of the Pennsylvania Railroad, of the Pennsylvania and Cambria Steel Companies.

Mr. Ely was a member of the Institution of Civil Engineers of Great Britain, the American Society of Mechanical Engineers, the American Institute of Mining Engineers, and an Honorary Member of the American Institute of Architects.

He was also deeply interested in science and art, and took an active part in many organizations for their advancement, among them being the American Society for the Advancement of Science, the American Philosophical Society, the Franklin Institute, the Drexel Institute, the American Academy in Rome, Italy, of which he was Vice-President, and the Philadelphia Orchestra Association, of which he was a Trustee.

Although ill health led to his retirement from active service with the Pennsylvania Railroad in 1911, it did not entirely deprive him of participation in the many affairs of life of which he was a part. His keen analytical and judicial mind, trained and broadened by his studies and an experience covering almost the entire period of railroad development, made him a valued adviser in all the activities with which he came in contact.

Mr. Ely held the degree of Civil Engineer from Rensselaer Polytechnic Institute, as a graduate, and had conferred on him the honorary degrees of Master of Arts, Yale, 1897, and Doctor of Science, Hamilton College, 1904.

Among the social organizations with which he was identified were the Philadelphia and the Merion Cricket Clubs, of Philadelphia; the University, Century, and Engineers Clubs, of New York City; and the Metropolitan Club of Washington, D. C.; the New England Society of Pennsylvania, and the Sons of the American Revolution.

Mr. Ely is survived by his four children—Mrs. Charles L. Tiffany, of New York City, the Misses Gertrude and Henrietta Ely, of Bryn Mawr, Pa., and Carl B. Ely, Assoc. M. Am. Soc. C. E., Superintendent of the Bridge Department of the Bethlehem Steel Company, Steelton, Pa.

Mr. Ely was elected a Member of the American Society of Civil Engineers on March 2d, 1881, and served as a Director in 1892-93.

RICHARD EVANS, M. Am. Soc. C. E.*

DIED DECEMBER 30TH, 1916.

Richard Evans was born at Caracas, Venezuela, on November 15th, 1855, and, on his father's side, came from the Evans family long established in New England. While he was quite young, his parents removed to the United States and settled in Woodbury, N. J. Here he received his preliminary education before entering the Polytechnic College of Pennsylvania, from which he was graduated with the Class of 1875.

In 1876, Mr. Evans was employed as Rodman and Levelman on the location of the Philadelphia and Atlantic City Railroad, and, from 1877 to 1879, he was engaged as Levelman and Transitman on topography and revision of lines and grades in the Twenty-third Ward for the City of Philadelphia. In 1879, he was appointed Transitman, and, later, Assistant Engineer on the preliminary and location surveys for the Danville and Shamokin Railroad, in Pennsylvania, and, in 1880, he was made Assistant Engineer on the location of the Five Mile Beach Railroad and laying out the Town of Anglesea, N. J. From July, 1880, to February, 1881, he served as Transitman in the Maintenance of Way Department of the Pennsylvania Railroad.

In 1881, Mr. Evans went to Mexico as Transitman and Chief of Field Party on the location surveys for the Mexican National Railroad, returning in 1882, to become Assistant Engineer on the New River Branch of the Norfolk and Western Railroad. In 1883, he was appointed Assistant Engineer with the Pennsylvania Schuylkill Valley Railroad Company, on the revision of part of its line. He designed and made the plans for arch culverts, piers, and bridge abutments on 6 miles of heavy construction, including the Schuylkill River crossing and viaduct approach.

From 1884 to 1887, Mr. Evans was engaged with the Survey Department of the City of Philadelphia on sewer construction in the Manayunk District and on laying out new streets in the Twenty-second Ward.

In 1887, Mr. Evans went West and was employed as Assistant Engineer with the St. Louis and San Francisco Railroad Company. Returning to the East in 1888, he was engaged until 1890 in private practice, making plans, estimates, and superintending the construction of macadamized and telford roads, highway bridges, etc., in Montgomery County, Pennsylvania. Afterward, he accepted for a short time a position as Principal Assistant Engineer on the Philadelphia and Seashore Railroad.

* Memoir prepared by the Secretary from information on file at the Society Headquarters.

In July, 1890, Mr. Evans established himself in private practice at Hagerstown, Md., in partnership with Mr. C. C. Vandevanter, which was continued until 1893. He then removed to Jamaica, N. Y., where he formed a partnership with his brother, Mr. C. A. Evans, and continued in private practice as a Consulting Engineer until his death, which occurred on December 30th, 1916. He was married in Philadelphia, Pa., about 1887, but his wife died two years later.

In his work and conduct, which was always above criticism, Mr. Evans exemplified the best influences of his profession. Realizing his incompatibility with the prevailing requirements of corporation service as he saw them, he preferred the private practice of engineering for which he felt himself better fitted temperamentally. Holding to that which he considered to be right, his decisions in regard to his work were made regardless of who was or was not benefited thereby. In the study of any contemplated project, he always showed that calmness, ease, and continuity of application which distinguish the trained engineer.

Mr. Evans was elected a Member of the American Society of Civil Engineers on June 7th, 1893.

FRANK FIRMSTONE, M. Am. Soc. C. E.*

DIED JUNE 27TH, 1917.

Frank Firmstone, the son of William and Mary Elizabeth Firmstone, was born on August 29th, 1846, at the Glendon Iron Works, near Easton, Pa., which his father had built in 1842. He received his early education at the Old Philips School in Easton. Later, he was sent to the Saunders Military Institute, in Philadelphia, Pa., where he was prepared for the Polytechnic College of Pennsylvania, and was graduated from that college as a Mining Engineer in June, 1865.

In November, 1865, Mr. Firmstone was employed as Levelman on surveys for the Wilmington and Brandywine Railroad, which position he held until March, 1866.

In January, 1867, he became associated with his father, as Assistant Superintendent, in the management of the Glendon Iron Works, remaining in this position until May, 1877, when his father died. Mr. Firmstone then took full charge of the Iron Works as General Manager, and retained this position until 1887, when he retired from active business. As Assistant Superintendent and as General Manager he made the designs and superintended the construction of all new work at the plant, including blast furnaces, etc., as well as repairs and alterations.

Subsequent to 1887, he became associated with the Cranberry Iron and Coal Company, serving for a number of years as its President and as a Director on the Boards of all its subsidiary companies until his death, which occurred at his home in Glendon, Pa., on June 27th, 1917.

He is survived by his brother, Mr. Harry Firmstone, of Longdale, Va.

Among other societies and clubs Mr. Firmstone was a member of the American Institute of Mining Engineers, and as such had contributed to its publications many interesting and valuable papers on blast furnace practice and on matters pertaining to the manufacture of pig iron, on which subject he was considered quite an authority.

He was also a member of the American Society for Testing Materials, American Forestry Association, Engineers' Club of New York City, Automobile Club of Philadelphia, Pa., Country Club of Northampton County, Pomfret Club of Easton, Pa., Northampton County Historical and Genealogical Society, and Trinity Protestant (Episcopal) Church, of Easton, Pa.

Mr. Firmstone was elected a Member of the American Society of Civil Engineers on August 7th, 1878.

* Memoir prepared by the Secretary from information on file at the Society Headquarters.

JAMES WALTER GRIMSHAW, M. Am. Soc. C. E.*

DIED FEBRUARY 15TH, 1917.

James Walter Grimshaw was born at Manchester, England, on April 19th, 1852. In 1868, he was sent to Germany where he was a pupil of Carl Bach, a well-known civil engineer of Stuttgart, until 1870. He then entered the machine shops of E. Harn, of the same city, where he served an apprenticeship of one year. In 1871, Mr. Grimshaw entered the Stuttgart Royal Polytechnic School, from which he was graduated in Civil Engineering in 1874.

After his graduation, Mr. Grimshaw returned to England, where he was employed until 1877 in the Engineering Departments of Messrs. Sharp, Stewart and Company and Messrs. Ormerod, Grierson and Company, of Manchester. He was also engaged for a short time as Resident Engineer of the Netham Chemical Works at Bristol.

In 1877, Mr. Grimshaw went to Australia where he spent the greater part of his professional life. He first entered the service of the South Australian Government and received the appointment of Resident Engineer in the Harbors and Jetties Department, where he remained until 1880. He then entered the service of the Government of New South Wales, in the Rivers and Harbors Department, his first position being Resident Engineer of the Sydney Water Supply Works, in charge of the construction and maintenance of the canals, tunnels, aqueducts, etc.

About 1890, Mr. Grimshaw made the surveys for the proposed water supply for the City of Armidale, and supervised the completion and testing of the steel service reservoir for the Albury water supply.

On the completion of this work, he was appointed Resident Engineer on the Richmond River and Harbor Works, where two breakwaters and a training wall were being constructed. Subsequently, he was sent to Sydney as Resident Engineer over the works in Sydney Harbor and the South Coast District. In this position he had charge of extensive wharf construction and the conversion of Darling Island into a deep-water shipping depot.

In 1901, Mr. Grimshaw retired from the service of the Government of New South Wales and returned to England, where he resided until his death which occurred at Brighton, on February 15th, 1917.

He was a member of St. Stephen's Club, of London, and of the Institution of Civil Engineers of Great Britain.

Mr. Grimshaw was elected a Member of the American Society of Civil Engineers on November 7th, 1888.

* Memoir prepared by the Secretary from information on file at the Society Headquarters.

WILLIAM OSWALD HENDERER, M. Am. Soc. C. E.*

DIED FEBRUARY 8TH, 1917.

William Oswald Henderer, the son of Myers Henderer and Euretta Curtis Henderer, was born on May 11th, 1865, in Greenbush, now known as Rensselaer, Rensselaer County, N. Y. He received his early education in the district schools of Schodack Landing and Castleton, in Rensselaer County, and in the grammar and high schools of Albany, N. Y. His technical training was received at Rensselaer Polytechnic Institute, Troy, N. Y., where he was a member of the Class of 1887.

From June, 1887, to January, 1888, Mr. Henderer served as an Assistant in the office of the City Engineer, of Troy. During the spring and summer of 1888 he worked on a preliminary survey through Northern Ohio for the Pennsylvania Railroad Company.

From May, 1888, until April, 1891, he was with G. W. G. Ferris and Company, of Pittsburgh, Pa., as Shop Inspector on bridge, railroad, and viaduct steelwork, and it was here that he became acquainted with Frank C. Osborn, M. Am. Soc. C. E., with whom, later, he was associated.

From April to November, 1891, Mr. Henderer was Draftsman and Assistant Engineer for the Detroit Bridge and Iron Works, and from November, 1891, to March, 1893, he served in a similar capacity with the Berlin Iron Bridge Company, of East Berlin, Conn.

In March, 1893, he went to Cleveland, Ohio, to be with Mr. Osborn, in charge of inspection and tests. From December, 1897, to June, 1900, as a member of the Osborn Company, Civil Engineers, he was in charge of structural design, inspection, and general office work. During this period, the Brooklyn-Brighton Viaduct and the South Rocky River Viaduct, in Cuyahoga County, Ohio, and the Zanesville Y-Bridge, in Zanesville, Ohio, were among the larger and more interesting works handled by the Company.

In June, 1900, The Osborn Engineering Company was incorporated, and Mr. Henderer became its first Vice-President. This position he held until the summer of 1910, at which time Mr. Osborn retired from active charge of the company, and Mr. Henderer became President.

In recent years, the Boston, New York, Cleveland, Washington, and Detroit baseball parks, and the factory buildings for the Firestone Tire and Rubber Company and The B. F. Goodrich Company, have been included among the more important structures designed by the Company.

* Memoir prepared by Kenneth H. Osborn, Assoc. M. Am. Soc. C. E.

Mr. Henderer continued as President of the Company until his death, on February 8th, 1917, at Miami, Fla. He had been absent from Cleveland only about 10 days on a fishing trip, when he was seized by a sudden attack of peritonitis. He was taken to a hospital in Miami, but lived only a few days after the attack.

Mr. Henderer was married on June 21st, 1893, to Ida F. Mayer, of Cleveland, and had one daughter, Geraldine.

He was an active member of the Cleveland Engineering Society, and had served as President of that organization. He was also a member of the American Society for the Promotion of Engineering Education, the Cleveland Athletic Club, the Shaker Heights Country Club, and the Chippewa Lake Club. He was deeply interested in the Masonic organizations of Cleveland, and was very active in all those of which he was a member.

Mr. Henderer was elected a Member of the American Society of Civil Engineers on April 3d, 1901.

ARTHUR HIDER, M. Am. Soc. C. E.*

DIED JULY 28TH, 1916.

Arthur Hider was born in London, England, on February 29th, 1844, and died at his home in Greenville, Miss., on July 28th, 1916. He was the son of James and Maria (Dear) Hider. In 1853, when he was nine years of age, his family moved from England to London, Ont., Canada. He received his early education in the schools of the latter place and afterward taught school there for a while. In 1864, at the age of twenty, he went to Cleveland, Ohio, where he began the study of engineering, as a private pupil, under Peter Emslie, a civil engineer, with whom he spent about one year.

In 1866, Mr. Hider was employed as a Draftsman by George Steuly, City Engineer of Louisville, Ky., and, after one year's service in that position, was appointed Assistant City Engineer and given charge of the public works in one-half of the city. After the death of Mr. Steuly, in 1869, Mr. Hider was continued in the same position under the late I. M. St. John, M. Am. Soc. C. E., who succeeded Mr. Steuly as City Engineer, until 1871, when the late Thomas P. Shanks, M. Am. Soc. C. E., became City Engineer. Mr. Hider was then appointed Principal Assistant City Engineer, and given charge of all improvements made by the city, except in the Sewer Department. He continued to serve in this position until some time in 1876, when Mr. Shanks was retired from office, and he, as the Principal Assistant, along with him.

From 1876 to the early part of 1879, Mr. Hider was engaged with his former Chief, Mr. Shanks, in engineering and contracting work, during which time they constructed a reservoir at Anchorage, Ky., and, for about one year, were engaged on masonry work at the new reservoir at Louisville, Ky., both of which were under the supervision of the late Charles Hermany, Past-President, Am. Soc. C. E., Chief Engineer of the Louisville Water-Works.

Mr. Hider entered the Government service in 1879, and, for a short time, was employed under Charles R. Suter, M. Am. Soc. C. E., Major, Corps of Engineers, U. S. Army (now Brigadier-General, *Retired*), in making hydrographic surveys and physical observations on the Mississippi River, near Arkansas City, Ark. Maj. Suter was appointed a member of the Mississippi River Commission, which body was created by Act of Congress, dated June 28th, 1879, and Mr. Hider was assigned to duty under Lieut. Smith S. Leach, Corps of Engineers, the Secretary of that Commission. From October, 1879, to November, 1880, he was engaged in making hydrographic surveys and collecting physical data on the river near Lake Providence, La., and, later, had charge of

* Memoir prepared by Charles H. West, M. Am. Soc. C. E.

a party taking slope observations along the river between Cairo, Ill., and New Orleans, La.

The Commission was now ready to begin construction, and, in September, 1881, Mr. Hider was assigned to duty as one of the Assistant Engineers in the Third Mississippi River District, extending from the White River to a few miles below the mouth of the Yazoo River. The Engineer Officer in charge of the district was Lieut. W. L. Marshall, who, later, became Chief of Engineers, U. S. Army. During the few years that Mr. Hider served under this officer, they learned to regard each other highly, and there ever after existed a friendly feeling between them.

At the time when Mr. Hider entered the Government service under the Mississippi River Commission, very few reliable data were available as to the regimen of the Mississippi, and few, if any, precedents of practical value as guides in a project of such character and magnitude as that which had been assigned to the Commission. His first service was in connection with surveys and observations to collect data necessary for a study and development of plans for the improvement of the river, but when it was decided to begin construction, he was transferred to that work, and soon became the Principal Assistant Engineer in charge of all revetment and other work of channel improvement in the Third Mississippi River District, serving continuously in that position until his death.

The first works of construction were necessarily experimental in type, as were also the various kinds of plant needed for carrying on the work. Mr. Hider was intimately connected with each step in the development of suitable types of construction, and also had much to do with the design and construction of towboats, hydraulic graders, and the various other items of plant required for the work in the Third District. He took great interest in the development of plans for the improvement and control of the river, and no small share of credit for successful work is due to his faithful, untiring, and patient attention to the duties of his position. He came to be regarded as an authority on bank revetment and other channel works, and his counsel in such matters always had weight.

Mr. Hider was connected with levee construction only in a limited way, but he was a close observer of what was going on in that line, and had very clear views in regard to that work.

In addition to his work under the Mississippi River Commission, he, occasionally, while on leave of absence and during spare time, served as Consulting Engineer in connection with municipal improvements, some of which were the design and construction of water-works systems at Greenville, Miss., Lake Providence, La., and other places; he was expert member of the Committee to determine the value of the water-works system of Memphis, Tenn., when that system was taken over by the City of Memphis.

Mr. Hider enjoyed remarkably good health until the last three years of his life. During the spring of 1913, while temporarily in charge of the work of closing the crevasse in the levee near Beulah, Miss., he was exposed to very inclement weather, with the result that he was taken seriously ill and never after entirely regained his health. He was unable to attend to active duties for more than a year prior to his death, but he continued to go to the office and, except during the last few weeks, he could be found at his desk every day when he was strong enough to leave home.

Mr. Hider was indeed a veteran of the Engineer Service of the Mississippi River. There being no provision in law for the retirement of civilian engineers in the service of the Government, he, like many others, remained on duty until advanced in age and literally worn out.

He lived to see nearly all the Engineer Officers under whom he had served in the early days on the river retired with high rank, and, as these men were near his own age and counted among his friends, he was always pleased to hear of any good fortune that came to any of them.

In later times, it was his province to serve under District Officers who were many years younger than himself, and to each of these he gave loyal assistance. They, in turn, recognized his ability and ripe experience, and his candid manner and earnest attention to duty never failed to win their confidence and esteem.

He was a good organizer, and believed in strict discipline. He recognized authority in those above him, and demanded obedience to orders from those under him. He was highly respected by his subordinates, and, though they dreaded his displeasure, they delighted in his approval, for he was just in his dealings with them and ever ready to give credit where it was due. It was a pleasure to him to advise and otherwise help along young engineers who appeared to be earnest and appreciative of his guidance.

Mr. Hider took an active interest in the civic affairs of the community in which he lived. There was no pretense and very little diplomacy in his nature, and he was outspoken in his opinions. He was well informed on many subjects other than his profession, and had a fine appreciation of music and art.

Mr. Hider was a member of the Protestant Episcopal Church, and though not outwardly a devout churchman, he had deep religious convictions. He loved the quietude of his home life, and was a devoted husband and kind father, and although he seemingly cared but little for social life in general, he was fond of the companionship of his friends.

He was married in Louisville, Ky., on October 6th, 1873, to Emma K. Anderson, who, with two daughters and three sons, survives him.

Mr. Hider was elected a Member of the American Society of Civil Engineers on September 7th, 1881.

SIDNEY WILLETT HOAG, Jr., M. Am. Soc. C. E.*

DIED NOVEMBER 1ST, 1916.

Sidney Willett Hoag, Jr., son of Sidney Willett and Ann Augusta Hoag, was born in New York City, on June 5th, 1857. He received his preliminary education in the public schools and was admitted to the College of the City of New York in 1872 as a student in the scientific course, from which he was graduated in June, 1877. From 1877 to 1879, he pursued the post-graduate course in Civil Engineering in the same college under the late Professor A. G. Compton, Assoc. Am. Soc. C. E., and was graduated with the certificate of Civil Engineer.

In 1879, after leaving college, Mr. Hoag went with the New York City and Northern Railroad, being first engaged on trestle construction. Subsequently, during construction, he had charge of the distribution of materials and supplies, and later was engaged in the construction of roadbed, turn-tables, and depots, from Croton Lake to Lake Mahopac.

In 1880 he entered the service of the City of New York, being engaged in the Department of Public Parks as an Assistant Engineer. His efforts in the Park Department were directed toward the public improvement of the annexed district. He had charge of various classes of engineering work under the jurisdiction of the Park Department, including surveys, laying out of streets, monumenting, preparation of tax maps, rule, damage, and benefit maps, and work of general construction involving regulating and grading, and, in addition, he directed the work of the various field parties.

In 1891, Mr. Hoag was appointed an Assistant Engineer in the Department of Docks and Ferries, and was assigned by George S. Greene, Jr., M. Am. Soc. C. E., then Engineer in Chief, as head of the drafting-room. In this position, Mr. Hoag directed the preparation of plans and specifications for various works of construction, undertaken by the Department along the water-front, such as piers, bulkheads, freight sheds, and freight and passenger terminals, and the approval of structural features of all plans submitted by private owners and lessees for all kinds of wharf structures.

In 1908, Mr. Hoag was appointed Deputy Chief Engineer of the Department under Charles W. Staniford, M. Am. Soc. C. E., Chief Engineer. During his incumbency as Deputy Chief Engineer, the Department of Docks carried out many important works of construction, involving piers, bulkhead walls, freight sheds, and ferry terminals. Some notable pieces of construction on which Mr. Hoag was engaged are the municipal ferry terminal at St. George, Staten Island, the municipal ferry buildings at the Battery, known as the Manhattan

* Memoir prepared by Charles W. Staniford, M. Am. Soc. C. E.

Terminal of the South Brooklyn Ferry, the Whitehall Terminal of the Staten Island Ferry, and also the viaduct approach to the St. George Ferry, at St. George. The most conspicuous and important undertaking by the Department of Docks during this period was the construction of the great transatlantic steamship terminal popularly termed the Chelsea Section. This is on the North River water-front, and covers the section from West 14th to West 23d Streets. It involved the construction of nine piers, with freight sheds thereon, and putting in heat, light, and power equipment for operating purposes. This section represents an expenditure of many millions of dollars, and the accommodation it affords for large transatlantic liners is not equalled anywhere in the world.

This large undertaking by the Department involved an enormous amount of detail work in connection with the preparation of plans and specifications, and the examination and approval of plans submitted by contractors during construction.

The successful completion of the Chelsea Section demonstrated Mr. Hoag's capacity for hard work, and his great attention to detail enabled the Department to carry out successfully many large undertakings involving extreme detail in steel construction and intricate problems in connection with heat, light, and power equipment. Mr. Hoag was a loyal assistant, conscientious in everything he did, and, above all, a man of sturdy integrity. His work was his hobby.

He never had any interest outside of his professional work, and was held in high esteem by the fellow members of his Profession.

In March, 1904, he became a member of the Society of Municipal Engineers of the City of New York, and at once took an active interest in its affairs. He was rewarded with honors for his labors, being elected Second Vice-President in 1910; First Vice-President in 1911, and President of the Society in 1912. He continued his interest in the Society, being on the Board of Directors as Past-President, at the time of his death.

In 1905, Mr. Hoag delivered, before the Municipal Engineers, a very interesting paper entitled "The Dock Department and the New York Docks." The paper dealt with the history of the Dock Department and its work, and contained considerable detailed data relative to water-front construction. It had a wide distribution, and was considered the most meritorious paper of the year. For this Mr. Hoag received the Society medal in recognition of his achievement.

In 1914, his health failed, and he was compelled by his physical condition to give up active work. He retired from the Department of Docks on a pension after 35 years in the service of the City.

After about a year's rest, he regained some of his strength and joined B. F. Cresson, Jr., M. Am. Soc. C. E., assisting the latter in the preparation of plans and specifications for the construction of

an extensive water-front and railroad terminal development at Bayonne, N. J. He was associated with Mr. Cresson up to the time of his death.

System, the love of detail, and the habit of accumulating interesting data pertaining to his Profession, were some of Mr. Hoag's personal characteristics. He was a thorough, painstaking, and conscientious worker, and the bigger the job, the happier it made him. Although, in his younger days, he was fond of athletics and had joined the New York Athletic Club, engaging in running races, his whole time, during his later years, was given to the work of his Profession. He was a capable executive, had a frank, genial personality, a high code of honor, and won the admiration of those who knew him.

Mr. Hoag was a member of the New York Athletic Club, the Municipal Engineers of the City of New York, and the Royal Arcanum.

Mr. Hoag was elected a Member of the American Society of Civil Engineers on September 2d, 1885. He served as a member of the Nominating Committee in 1913-14.

CHARLES WILCOX HOTCHKISS, M. Am. Soc. C. E.*

DIED OCTOBER 28TH, 1916.

Charles Wilcox Hotchkiss, the son of Edgar F. and Caroline Enos Hotchkiss, was born at Unadilla Forks, N. Y., on June 19th, 1863.

Mr. Hotchkiss began his engineering career as a Rodman on the New York, West Shore, and Buffalo Railroad, when he was 20 years old. He left that position after a short time to enter the employ of the Pennsylvania Railroad Company as Rodman and Assistant Engineer, which position he retained until August, 1886, when he went to the Michigan Central Railroad, as Assistant Engineer in charge of construction. In 1890, he was appointed Division Engineer in charge of the Chicago Division, and, in May, 1895, he became Chief Engineer of the Chicago, Hammond, and Western Railroad, in full charge of all matters in connection with its construction. In 1897, Mr. Hotchkiss held the position of Chief Engineer and Superintendent of Construction for the Chicago, Hammond, and Western Railroad Company, and of the Michigan Central Terminal Railroads, in charge of the extension of the Chicago Terminal. In 1899, he was appointed Chief Engineer of the Chicago Transfer and Clearing Company, and Consulting Engineer of the Michigan Central System.

In 1900, Mr. Hotchkiss became President of the Indiana Harbor Railroad Company, and constructed the Indiana Harbor Belt Railroad from Indiana Harbor around Chicago, as well as the Chicago, Indiana and Southern Railroad, from Chicago to Danville, Ill. After completing these roads in 1905, he effected their consolidation with the New York Central System, and became General Manager of the consolidated companies.

In 1912, Mr. Hotchkiss became President of the Chicago Utilities Company and the Chicago Tunnel Company, and, in 1913, he was called to New York City to take over the management of the various properties and interests of the late H. H. Rogers. At the time of his death, which occurred at Battle Creek, Mich., on October 28th, 1916, from organic heart trouble, he was Chairman of the Board of Directors of the Virginian Railway Company; Chairman of the Atlantic Coast Electric Railway Company; President of the Chicago Tunnel Company, the Rail Joint Company, the Richmond Light and Railroad Company, the Staten Island Midland Railway Company, and of various other power, light, and railroad companies.

Mr. Hotchkiss was a pioneer in the promotion and development of the Calumet Manufacturing District of Indiana, particularly in the Cities of East Chicago, Indiana Harbor, Hammond, and Michigan

* Memoir prepared by the Secretary from information on file at the Society Headquarters.

City. By his construction and development of the railroad facilities of that District he laid the groundwork for, and made possible, the present tremendous growth of these localities. He retained an active interest in the Calumet District until his death, and the recently completed building for the First Calumet Trust and Savings Bank, at East Chicago, which is the finest bank building in Northern Indiana and was erected under the personal direction of Mr. Hotchkiss, together with the Indiana Harbor Belt Railroad, are the two outstanding monuments to his activities in that region. He also had large real estate holdings there, extending from Laporte and Michigan City, Ind., to Chicago, Ill.

Mr. Hotchkiss is survived by his widow, Mrs. Mary Jayne Hotchkiss, and by two brothers and one sister.

He was a member of the Chicago, Mid Day, Press and Engineers' Clubs, and of the Indiana Society, of Chicago, Ill.; Duquesne Club, of Pittsburgh, Pa.; the Lawyers', Engineers', Transportation. and Richmond County Country Clubs, of New York City; the Western Society of Engineers; the American Railway Engineering Association; and a Life Member of the Art Institute of Chicago. He was also a member of the Protestant Episcopal Church and the Medinah Temple Shrine.

Mr. Hotchkiss was elected a Member of the American Society of Civil Engineers on January 5th, 1898.

WILLIAM HENRY HUNTER, M. Am. Soc. C. E.*

DIED FEBRUARY 27TH, 1917.

William Henry Hunter, the son of Henry Hunter, was born at Cork, Ireland, on June 16th, 1849. While he was quite young, his parents removed to England, and the boy was educated at private schools at his home in Sunderland and at the College of Physical Science, University of Durham, at Newcastle-on-Tyne.

In 1866, he entered the Engineering Workshops of the River Wear Commissioners where he remained until 1868, when he went into the office of the late Thomas Meik as a pupil.

Having served his pupilage, Mr. Hunter, in January, 1870, was employed as Assistant in the office of Messrs. Meik and Nisbet, Harbor and Dock Engineers, of Edinburgh and Sunderland, and was engaged on harbor and dock work on the Rivers Wear, Blyth, and Coquet, at Burntisland and Ayr, and also on surveys for the Hylton, Southwick, and Monkwearmouth Railway, a branch of the North Eastern Railway.

From January, 1872, to September, 1873, he served as Resident Engineer on the construction of the Hylton, Southwick, and Monkwearmouth Railway in the County of Durham. He then accepted the position of Assistant Engineer on the reconstruction of the River Weaver Navigation, in the County of Chester, and remained in that position until 1882. The work consisted of enlarging and adapting the River Weaver for navigation by large barges and coasting vessels, thus giving commercial outlet to the interests engaged in the Cheshire salt trade.

From 1882 to 1885, Mr. Hunter was engaged as Chief Assistant to the late Sir Edward Leader Williams, on the surveys, preparation of designs, and Parliamentary contests, in connection with the project to connect the City of Manchester with the sea by a ship canal, and on work for the regulation and improvement of the River Dee in Cheshire and North Wales.

In 1885, when the Manchester Ship Canal Company was incorporated, Mr. Hunter was appointed Assistant Chief Engineer, and, as such, was directly and responsibly concerned with all the engineering operations connected with the actual design and construction of that waterway, including the Canal proper and all its auxiliary works.

In 1895, he was made Engineer and, in 1896, Chief Engineer of the Manchester Ship Canal, which position he held until 1910. During this time Mr. Hunter had charge of the maintenance of the entire project and carried out all additional works necessary to its development, including the extension of the great system of docks, railways,

* Memoir prepared by the Secretary from information on file at the Society Headquarters.

etc. It was during his incumbency as Chief Engineer, that the new Dock No. 9 was opened, with great ceremony, by His Majesty, the late King Edward VII, on July 5th, 1905.

In 1910, Mr. Hunter was appointed Consulting Engineer of the Manchester Ship Canal, which position he held until his death, which occurred at his home at Bank House, Woodley, Cheshire, on February 27th, 1917.

Mr. Hunter was recognized as the foremost English authority on canal and harbor construction, equipment, and operation. In 1898, he was appointed a member of the Comité Technique which was constituted by the French Government to consider and prepare plans, for the New Panama Canal Company, for the French Panama Canal, and, in 1905, at the request of the United States Government, he became a member of the Advisory Board of Consulting Engineers, appointed by the President to consider and report on the plans for the American Panama Canal project, for which he strongly favored a sea-level, as against a lock, canal.

Mr. Hunter was a member of the Institution of Civil Engineers and frequently took part in the discussion of papers on canals and waterworks. He also presented a paper on the "Artificial Waterways in Great Britain",* before the International Engineering Congress, held in St. Louis, Mo., in 1904, under the auspices of the Society.

He was also a member of the Society of Arts and the Manchester Association of Engineers, and a Commissioner for the Navigation for the Upper Mersey. Mr. Hunter was of a deeply religious nature and was a devoted member of the Plymouth Brethren, frequently preaching and taking an active part in their meetings in Lancashire and Cheshire. A strenuous business man, he was noted for his gentleness and courtesy as husband, father, and host, and his love for children and their love for him was a frequent cause of remark. He is survived by his widow and three children.

Mr. Hunter was elected a Member of the American Society of Civil Engineers on February 7th, 1906.

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* *Transactions*, Am. Soc. C. E., Vol. LIV, Part F, p. 183.

JAMES EDGAR JENKINS, M. Am. Soc. C. E.*

DIED JULY 5TH, 1917.

James Edgar Jenkins was born in Vernon, N. Y., on November 25th, 1878. He received his early education in the Vernon schools, and was graduated from the village High School in 1896.

His father, Josiah W. Jenkins, is a well-known surveyor and hydraulic engineer, and from him the boy learned the rudiments of engineering so well that he qualified as Rodman with the United States Deep Waterways Survey, on which work he was engaged during 1897-98.

In 1898, the Utica Gas and Electric Company undertook the hydro-electric development of West Canada Creek, at Trenton Falls, N. Y., and, for nearly three years, Mr. Jenkins was Instrumentman and Inspector on this project.

In the mid-term of 1900, he entered Rensselaer Polytechnic Institute where he quickly overtook his class, and was graduated in excellent standing with the degree of C. E. in 1904. During his summer vacations he was employed as Instrumentman on the Rutland Canadian Railroad and the Fort Plain, N. Y., Water-Works.

Immediately after his graduation, Mr. Jenkins entered the employ of the engineers in charge of the Pennsylvania Railroad Tunnels, in New York City, as Rodman, and shortly became Chief of Alignment on the East River Division. This led to a position as Engineer with the Naughton Company and Arthur McMullen, on Sections A and B.

For the next three years he was Engineer for the Thomas Crimmins Contracting Company, on Barge Canal Contract No. 9 and on several minor contracts in New York City.

Mr. Jenkins was then employed as Engineer for Smith, Hauser, Locher and Company, on Aqueduct Contract No. 66, which position he retained for nearly three years. While engaged on this work his health began to fail, due to pulmonary trouble, and he sought to regain it by a stay at Southern Pines, N. C. This sojourn was shortened, in order that he might undertake the alterations of the dam, spillways, etc., of the Union Manufacturing and Power Company, on Broad River, South Carolina.

Shortly thereafter, in March, 1915, Mr. Jenkins became Superintendent for the Cook Construction Company, on Contract No. 2 of the Montreal Aqueduct Enlargement. When this work was closed down by war conditions, he returned to New York and, after a brief rest, took charge of the erection of a large concrete chemical mill at

* Memoir prepared by H. R. Beebe, Assoc. M. Am. Soc. C. E.

Buffalo for the contractors, the John W. Cowper Company. This contract was carried out during the winter of 1916-17, and on the completion of the work Mr. Jenkins' health was so precarious that he returned to his home in New York to recuperate. Shortly afterward, however, he became associated with the writer in constructing a roundhouse at North White Plains, but, before its completion, he was obliged to leave for a sanitarium at Saranac Lake, N. Y., at which place he died of tuberculosis, on July 5th, 1917.

From the foregoing summary of his activities, it will be noted that Mr. Jenkins was never idle. In his spare moments, he was a frequent visitor to the Society House, and had a large acquaintance among the members. He was of an inventive turn of mind, and had patented several devices along mechanical lines, including an automatic flashboard and a snow-melting furnace. This furnace he had intended to try out on a commercial scale during the coming winter, and it was expected that it would be so efficient that large numbers would be used in competition with the ordinary methods of snow removal.

Mr. Jenkins' early death was due in large part to his unwillingness to recognize the signs of failing health. He seemed to have a conviction that his body was a machine which, with minor repairs, could be run indefinitely, and that overhaul could be put off safely until his complete convenience. In spite of ill-health, his mind and judgment were not affected, and his cheerfulness was the admiration of all who knew him. That his career was cut so short is a source of deep grief to his friends and to those acquaintances who recognized in his professional attainments the promise of a bright future.

In June, 1907, Mr. Jenkins was married to Miss Helen Day, of New York City. He is survived by his widow and two children, a girl of seven and a boy of five, and also by his parents, Mr. and Mrs. J. W. Jenkins, of Vernon, N. Y., and one sister, Mrs. Carl H. Dudley, of Silver Creek, N. Y.

Mr. Jenkins was elected an Associate Member of the American Society of Civil Engineers on December 5th, 1906, and a Member on March 14th, 1916.

WALTER KATTÉ, M. Am. Soc. C. E.*

DIED MARCH 4TH, 1917.

Walter Katté was born in London, England, on November 14th, 1830. He was the son of Edwin Katté, and the grandson of Edwin Katté, a political refugee from Prussia during the reign of Frederick the Great. His mother, Isabel Chambers, was the granddaughter of John Chambers, a celebrated boat builder on the Thames, London. Walter Katté was educated in Kings College School, London, and, after his graduation, spent three years as an apprentice in the office of a civil engineer.

In 1849, he came to the United States and entered American railroad service as Clerk and Draftsman for the Chief Engineer of the Central Railroad of New Jersey, from Whitehouse to Easton, Pa. Later, he served as a Rodman and Assistant Engineer on the Belvidere and Delaware Railroad. In the early Fifties, he acted as Engineer for a land development company, and laid out the Town of Deerman, now Irvington-on-Hudson, N. Y.

During the three years following 1854, Mr. Katté was Chief Assistant Engineer on the Western Division of the Pennsylvania Railroad. Later, he acted, successively, as Resident Engineer of the Pennsylvania State Canals, as Assistant Engineer of the Pittsburgh, Fort Wayne and Chicago Railroad, and the Pittsburgh and Steubenville Railroad, until the breaking out of the Civil War.

In 1859, he was married, in Greensburg, Pa., to his first wife, Margaret Jack, who died in 1864, leaving one son.

During 1861 and 1862, Mr. Katté served as a Colonel of Engineers in the Union Army, being commissioned from civil life to a staff position. He was assigned to bridge work in Washington, D. C., and at various points in Virginia and Maryland. He was the engineer in charge of the construction of the so-called "Long Bridge" over the Potomac River at Washington. While in this position, he had an experience with the great cavalry leader, General Philip Kearny. Returning one night from Washington to his regiment, which was quartered across the Potomac, the General sent his orderly ahead to demand passage over the bridge and received word that it was not in condition for traffic. He immediately rode his horse at full speed on to the bridge and, on being stopped peremptorily by Col. Katté, demanded an explanation as to why he was not permitted to proceed. Col. Katté quietly explained that a gap of 100 ft. or more in the bridge structure

* Memoir prepared by George W. Kittredge, M. Am. Soc. C. E.

would prevent his further progress, except by swimming. Commenting editorially, a New York daily paper said:

"In the Civil War nothing was more important to the safety of Washington than keeping the railroads and bridges in its neighborhood in perfect shape. This work Col. Katté supervised efficiently. Though not won on the battlefield, his military title stood for active, energetic work in the interests of military defense."

In 1863, Col. Katté was engaged as Chief Engineer of the Lewiston Branch of the Pennsylvania Railroad, and, later, as Resident Engineer and Engineer of Bridges and Buildings on the Northern Central Railroad, from Baltimore, Md., to Elmira, N. Y.

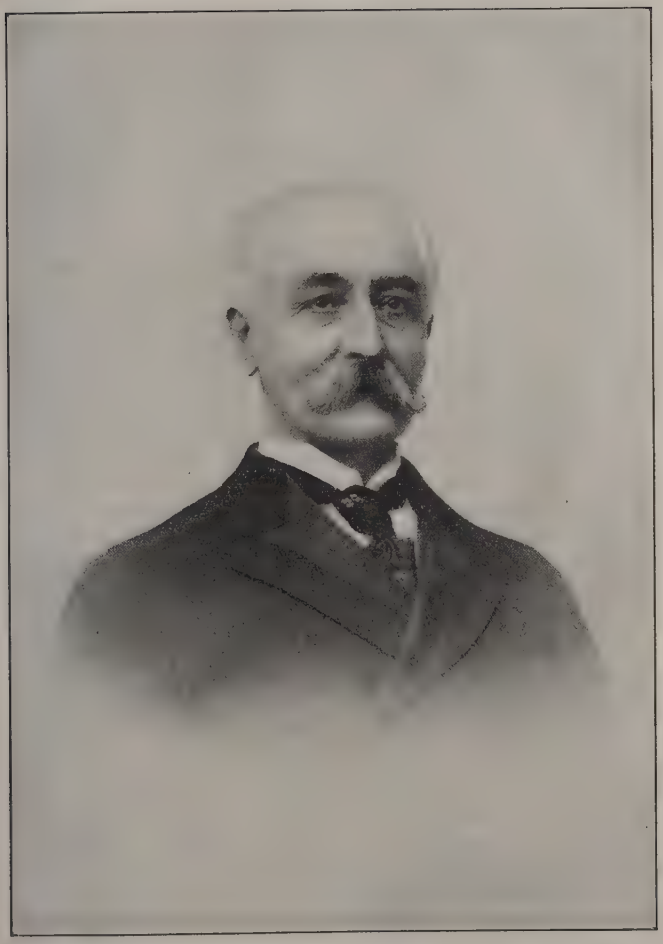
Col. Katté compiled and wrote the first "Carnegie Pocket Companion" published, and, recently, at the request of the Carnegie Steel Company, wrote the history thereof, as follows:

"In 1865 to 1868, I was resident in Pittsburgh as Engineer and Secretary of the Keystone Bridge Company. In 1868, that company and the Union Iron Mills of Pittsburgh (Carnegie Bros., Kloman, Phipps & Co.) decided to enter the western field in competitive business and to establish an office and representative in Chicago for that purpose; I was chosen for that position. The Keystone Bridge Company had at that time already under contract the manufacture and erection of the superstructures of the Hannibal & St. Joe Railroad Company's bridge over the Missouri River, at Kansas City, and the Illinois Central Railroad Company's bridge over the Mississippi River, at Dubuque, Ia., also was negotiating contracts for bridges, later consummated, for Mississippi River at Keokuk, Ia., Louisiana, Mo., and St. Louis, Mo.

"I proceeded to Chicago and opened there the western office of the Keystone Bridge Company and the Union Iron Mills, of Pittsburgh, Pa., and took personal charge, as agent and representative, of the field operations under these contracts.

"In 1870, negotiations for the great steel arch bridge over the Mississippi River at St. Louis were being actively promoted by Mr. Andrew Carnegie and myself, and finally consummated in the execution of a contract, signed by Capt. James B. Eads, as President of the Illinois & St. Louis Bridge Company, and by myself, on the part of the Keystone Bridge Company, under the terms of which, the Keystone Bridge Company undertook and obligated itself to perfect the mechanical details of the shop drawings of the superstructures, supply all materials for, and manufacture of, same, design plans for erection, and to erect it and assume all responsibility for the successful completion of the erection.

"I was assigned to take personal charge, as Resident Engineer, of said erection. As the responsibility for the successful consummation of same was of extreme gravity, I felt the paramount necessity of my personal presence on the work continuously, which, of course, resulted in the closing of my office in Chicago and the removal of same to St. Louis, which was effected early in 1871, and the joint Western office of the Keystone Bridge Company and the Union Iron Mills, of



WALTER KATTÉ

Pittsburgh, Pa., was opened under my charge at No. 211 Washington Avenue, St. Louis, Mo.

"About this time, or a little later, Mr. Thomas Carnegie suggested to me his desire to issue a handy 'Pocket Book' as a desirable assistant to Engineers and Architects in making proper selections, suited to their requirements, of the various products of the Union Iron Mills and asked me to compile the mss. for it, which I did. It was all written by my own hand from time to time in such leisure moments as were available, notwithstanding the pressing demands of my every day work, most of it done at home in evenings—and that's about all the early history that this little progenitor has to claim. It proved, however, a great success when issued, and there was, so Thomas Carnegie told me, a great demand for it, and he wrote me that he had received many letters from Engineers and Architects using it—highly extolling its usefulness and wondering why such a handy little *vade mecum* had not been issued long before."

While living in St. Louis, Col. Katté was married to Elizabeth Pendleton Britton, daughter of the Hon. James H. Britton, a prominent banker and later Mayor of that city.

After the completion of the St. Louis Bridge, Col. Katté was called to New York City to take the position of Chief Engineer of the New York Elevated Railroad Company, and from 1877 to 1880, he built the initial portions of the Third Avenue and Ninth Avenue Elevated Railroads, which were the first elevated steam railroads.

His next work was the construction of the New York, Ontario and Western Railroad, from Weehawken, N. J., to Middletown, N. Y.; then the building of the West Shore Railroad from New York City to Buffalo, which was followed by the construction of the Jersey Junction Railroad, connecting the West Shore Railroad with the Pennsylvania Railroad, at Jersey City. This work occupied his time between 1880 and 1886.

In 1886, Col. Katté became Chief Engineer of the New York Central and Hudson River Railroad Company, which, at that time, absorbed the West Shore Railroad. His most important work while in the employ of this Company was the four-tracking and depressing of the tracks, in New York City, north of the Harlem River, this work being known as the Harlem Depression; the construction of the four-track steel viaduct in Park Avenue, New York City, and the four-track drawbridge over the Harlem River, which is still the largest drawbridge in existence. In 1898, Col. Katté resigned his position with the New York Central Company and, in his letter of resignation, stated: "The recent absorption of other railroad lines into the Vanderbilt System had so multiplied the duties of the office of the Chief Engineer, that he felt that a younger man was necessary for the work." In accepting his resignation, the Hon. Chauncey M. Depew, then President of the Railroad Company said: "Col. Katté is one of the fore-

most engineers in the world. He is still connected with the New York Central and Hudson River Railroad Company as Consulting Engineer, and will be as long as he lives."

Col. Katté was one of the original thirteen founders of the Western Society of Civil Engineers of which he only recently was elected an Honorary Member. He was also a member of the Institution of Civil Engineers, of Great Britain.

During his active engineering life, Col. Katté made frequent contributions to technical papers and to the *Transactions* of the National Engineering Societies. He published one of the first sets of standard specifications for railroad construction work, and had taken out several U. S. Patents, the one in most general use being his so-called "Three-tie rail joint."

Important daily papers at the time of Col. Katté's death were unanimous in their expression of the fine quality of his engineering work, one commenting editorially as follows:

"Col. Katté was a fine American, a great railroad uilder, and had won first place among our civil engineers * * *. He knew and cared little about the devious ways of financing railroads; everything about construction and operation. Half a century of such activity fairly earned a period of repose. Col. Katté's later years were peaceful, calm, uneventful. He will live in the memory of his profession as a man who saw things clearly and who did things thoroughly. That is, from the practical viewpoint, the highest of encomiums."

Col. Katté enjoyed nearly nineteen years in quiet retirement. His health was excellent, with the exception of almost total deafness; his mind alert and vigorous; his spirit strong and serene until the day of his death. He died at his home, in New York City, on March 4th, 1917, and is survived by a widow, two sons, and a daughter.

Col. Katté was elected a Member of the American Society of Civil Engineers on October 7th, 1868, and served as a Director in 1885 and 1889.

WILLIAM BYRD KING, M. Am. Soc. C. E.*

DIED OCTOBER 11TH, 1915.

Mr. King was born in Orange County, Virginia, on December 29th, 1848, and was a direct descendant of Colonel William Byrd, who founded Richmond in 1733.

His early education was through tutors and private neighborhood schools. Later, he took academic work at Locust-Dale College and some engineering courses at the University of Virginia.

In 1873 Mr. King went to Texas, where he engaged in land surveying, principally the demarcation of railway land grants in the Far West. He continued this pioneer work, under all sorts of hardships and dangers, until 1879, when he was appointed Assistant Engineer on Construction of the Texas and Pacific Railway, and later of the Fort Worth and Denver City Railway, and the Missouri, Kansas and Texas Railway, in Texas.

In 1884 Mr. King was appointed City Engineer of Fort Worth, serving until 1889, when he was appointed City Engineer of Waco, serving until 1891. During his incumbency of these two offices he planned street and sewerage systems that were basic for both municipalities.

After completing his work at Waco, Mr. King was called to the Chief Engineership of the Fort Worth and Rio Grande Railway, now a part of the Frisco System. Later, he was elected Vice-President and General Manager of the Frisco System in Texas, which position he filled with distinction until Armour and Swift established their great packeries and stockyards in Fort Worth, in 1902, when he was made President of the Fort Worth Belt Railway, and Vice-President and General Manager of the "Armour-Swift" Stockyards.

For several years Mr. King was a sufferer from arterio-sclerosis, and in 1915 was retired, with consultation duties only, on full salary. For perhaps two years before this he had suffered from the insomnia characteristic of his malady to the extent that 10 or 12 hours' sleep a week was his gauge of rest. In spite of this, he was regularly at his desk, the suave, courteous, directing head of the busiest cattlemart of the Southwest.

In addition to his other engagements, he found time during 1913 to act as a member of the Board of Engineers in charge of Fort Worth's new water supply. His work on that Board helped to bring to light and eradicate one of the most insidious pieces of municipal graft ever known in the South.

* Memoir prepared by John B. Hawley, M. Am. Soc. C. E.

During several of the quite strenuous Board meetings on this matter, he was obliged to retire to his physician's office and submit to copious bleeding, in order to keep his blood pressure within living bounds. Little things like letting out part of his heart's blood in order to stand the strain of serving the public did not seem to "cut any figure".

The writer knew "Byrd" King, with increasing intimacy, from 1891 until his death. He was the kind of man we all want really to have as a friend. An Engineer of the Tredgold type. A typical Southern gentleman? No. Byrd King's courtly manners marked him as the son of many generations of Virginia Cavaliers, but his gentleness went far beyond any qualification as to South or North: his innate, never spoken, but always to be depended on, rule of action was, *noblesse oblige*.

William Byrd King was a world gentleman.

Mr. King was elected a Member of the American Society of Civil Engineers on October 7th, 1896.

EUGENE CASTNER LEWIS, M. Am. Soc. C. E.*

DIED FEBRUARY 13TH, 1917.

On the left bank of the Cumberland River, about eight miles above Dover, the county seat of Stewart County, Tennessee, there may still be seen the ruins of the once famous Cumberland Iron Works, which were destroyed by the Federal Army after the capture of Fort Donelson, on February 16th, 1862.

In 1830, two brothers, John and Samuel Stacker, came from Pennsylvania to Tennessee, and in beginning this enterprise became the founders of the iron industry in this section of the State. With them came George T. Lewis, then a lad sixteen years of age. Thirty-two years later, this property had developed into a great domain of more than 60 000 acres, from which were obtained the ore, limestone, and charcoal used in its furnaces. The Hon. John Bell, James Woods, and the Yeatmans had become interested, and George T. Lewis was General Manager of the works and the affairs of the partnership. The chief product of the foundry was the massive iron kettles used in the manufacture of sugar in the Southern States and abroad. The furnaces supplied the charcoal, pig iron, and blooms used in the foundry and rolling mill, the surplus being sold on the market. The product of the rolling mill was bar iron of exceptionally high quality. About 500 slaves and many employed men supplied the labor.

Mr. George Thomas Lewis, the efficient Manager of this great property, and his wife, who was Miss Margaretta Barnes, of Philadelphia, Pa., were the parents of six children, all born at their home at the Cumberland Iron Works. The fourth child, born June 22d, 1845, was Eugene Castner Lewis, the subject of this memoir.

Born and reared in the very midst of organized and productive labor, the young lad inhaled with the air that he breathed the instinct and inspiration that abided with him through all the stirring years of his life. His motives and purposes were ever for production and construction; never for the reverse.

At his father's home, and at a private school in the neighborhood, he learned the rudiments of education. This was followed by a course at Stewart College, in Clarksville. Then came the four years of war between the States. Fort Donelson, only nine miles from his home, was the scene of fierce fighting and final capture. The gunboats came up the river, and his father, bound and taken as a prisoner on one of them, was forced to face and witness the destruction of the plant, the work of his lifetime. Thrilled with anger and indignation, the lad,

* Memoir prepared by the following Committee: Wilbur F. Foster, Hunter McDonald, John Howe Peyton, and Richard Montfort, Members, Am. Soc. C. E.

Eugene, was so unguarded in his talk, so eager to help the Confederates, that he was threatened with imprisonment. This, however, was waived on condition that he be sent out of the State; and thus it was that Eugene became a student at the Pennsylvania Military Academy, then at West Chester, Pa., from which he was graduated with the Class of 1865 as one of ten, most proficient in mathematics and kindred studies. Here, also, he acquired a title, due perhaps to his proficiency, popularity, or characteristic leadership, a title that stayed with him to the end of his days, and for more than fifty years he was only and always known as "Major Lewis." No other title seemed to fit.

With very brief delay after his graduation from the Military Academy, Major Lewis began his professional career, in October, 1865, in the service of the Memphis, Clarksville, and Louisville Railroad, as Assistant Engineer, with the late William D. Pickett, Hon. M. Am. Soc. C. E.

In May, 1866, he entered the service of the Louisville and Nashville Railroad, as Assistant with the late Rudolph Fink, M. Am. Soc. C. E., in the Road and Engineering Department, and, from October, 1869, until November, 1870, he was in charge of the construction of the Glasgow Branch of that road.

He was then engaged in making surveys for three projected lines of railroad in Western Tennessee and Northern Mississippi until July, 1871, when he was placed in charge of the construction of the road which is now the Owensboro and Russellville Division of the Louisville and Nashville Railroad.

On completion of this service, Major Lewis withdrew from strictly professional engagements as a Civil Engineer; and, yet, in the multifarious affairs of his active life, his education for the Profession, together with his native talent, experience, and close study of engineering work, constituted a most important factor in his equipment for the conception, organization, and successful accomplishment of the many notable achievements in which his skill and training as a Civil Engineer were in constant service.

As the business member of the firm of W. B. Read and Company, Major Lewis was engaged, in 1873-74, in the execution of a contract for the construction of certain sections of very heavy work in Kentucky, on the railroad which is now a part of the Queen and Crescent System.

Through the large purchase of explosives used on this work, Major Lewis became favorably known to the Dupont Powder Company. This led to the acquaintance and personal friendship of its owners, and to his appointment as their agent for the sale of the product of their subsidiary mills at Sycamore, in Cheatham County, Tennessee, and,

later, to his entire control and management of that property, until, in 1904, manufacture at that plant ceased, and the mills were dismantled.

Sycamore Mills was the business headquarters and residence of Major Lewis from 1875 to 1897, and, during the summer, until 1907. The memory of his public spirit and usefulness is cherished and honored in that community, where the many improvements in highways, bridges, and other details still bear testimony to his skill, energy, and good judgment. It was in the reconstruction of one of these bridges that the accident occurred which finally cost him his life.

It was during the period of his engagement in this enterprise that Major Lewis was married, on October 12th, 1880, to Miss Pauline Dunn, of Nashville, Tenn., and, at their home at Sycamore, seven children were born, all of whom are living. Mrs. Lewis died at Nashville, on January 14th, 1902.

In the executive department of railways, Major Lewis served with great ability and success: On December 8th, 1896, he was elected a member of the Board of Directors by the Stockholders of the Nashville, Chattanooga, and St. Louis Railway, and served faithfully and efficiently in that capacity until his death. He was Chairman of the Executive Committee from December 14th, 1898, to October 13th, 1914, and was "Chairman of the Board" from February 28th, 1906, until his death. Twice the duties of Acting President were added to his other service; first, from February 17th to February 28th, 1906; and, again, from December 19th, 1913, to April 1st, 1914.

For more than 30 years, Major Lewis possessed the friendship and confidence of the chief officials of the Louisville and Nashville Railroad System, and by them was entrusted with the care of its interests in many matters of importance, especially in its affairs, and the development of its facilities in Tennessee. An important item in this service was the following:

"The Louisville and Nashville Terminal Company" was chartered on March 22d, and organized on March 28th, 1893, for the purpose of perfecting joint terminal facilities for the use of the Nashville, Chattanooga, and St. Louis Railway, and the Louisville and Nashville Railroad, and the erection of the necessary buildings required for the joint or separate use of those roads.

The steps preliminary to the purposes of this charter were all under the personal direction of Major Lewis, and extended over a period of several years, required for the purchase of the necessary real estate. Major Lewis was made President of the Company, on March 2d, 1898, and served in that office until November 27th, 1905. In that capacity, he had responsible control of every detail, including the approval of all plans and designs, and the awarding of contracts; also, the general supervision of grading the grounds, construction of the

Union Passenger Station and other extensive buildings, steel viaducts, bridges, yard trackage, and other details, altogether involving an expenditure of more than $2 500 000. This work was begun in August, 1898, and formally opened for service in October, 1900.

The Nashville and Decatur Railroad, beginning at Nashville, Tenn., and ending at Decatur, Ala., is under lease for a long term of years to the Louisville and Nashville Railroad, but the organization of the Lessor Company is maintained, and, of this, Major Lewis was a member of the Board of Directors, and Vice-President at the time of his death.

The Napier Iron Works are in Lewis County, Tennessee, and, from the time of its organization in 1890, until 1895, Major Lewis was President of the Company and General Manager of the works. A branch line of the Louisville and Nashville Railroad was built from Summertown to Napier, and the furnace plant at that place was built and operated as a charcoal furnace until 1895 under his management.

By the Courts, Major Lewis was entrusted with the following responsible duties:

In 1901, he was made Co-Receiver in the affairs of the Nashville Street Railways. This resulted in the re-organization of the Company under the title of The Nashville Railway and Light Company, in 1903.

In June, 1910, he was appointed Co-Receiver in the affairs of the Bon Air Coal and Iron Company. In the performance of this trust, he continued until his death.

It is probable that his notable success in the organization, development, and management, as Director General, of the Tennessee Centennial Exposition, held at Nashville in 1897, is the especial achievement by which Major Lewis was most widely known, and for which he justly received the greatest commendation from the public.

It is impossible, in this memoir, to give adequate or even partial description of this memorable and historic event and its far-reaching influence for good. Suffice it to say that it was the unanimous and unqualified action of the large and very able Board of Directors, as well as the voice of the people at large, that the complete success of the Exposition was, in great measure, due to the skill, inventive genius, and admirable taste of Major Lewis in the design and construction of details, and to his rare administrative ability in the management of its affairs.

As a natural consequence, and under the inspiration of the example set in the attractive beauties of the Exposition grounds, there came the creation of the Park System of the City of Nashville; and from 1901 to 1913, Major Lewis was the moving spirit, the presiding genius of the Board of Park Commissioners. Evidence of his taste, skill, and originality is seen in numberless det ils, sometimes unique,

always attractive, in the many Parks of Nashville, to which he devoted much time and study. At the Centennial, "The Parthenon", an accurate reproduction of the original at Athens, still stands as a memento of the Exposition.

The Nashville American had long been the leading morning daily newspaper of the city. In 1896, Major Lewis became its owner and publisher, and continued as such until 1909. Under his management, it was a vigorous and influential advocate of the policies of the political party of which it was the organ, and of integrity and "up-to-date" methods in municipal affairs. One especial reason for its purchase in 1896 was for its service in the Publicity Department of the Centennial Exposition.

The Engineering Association of the South was organized at Nashville, on September 18th, 1889, and Major Lewis was one of the charter members. He served as Director for several terms, and was President of the Association in 1893.

The following are some of the personal attributes of Major Lewis, which combined in the formation of a character, aggressive, public spirited, and of conspicuous usefulness in public affairs and in private life: Strict integrity and faithfulness in official station, and in matters of public or private trust; and, in this, he was imperative in demanding the same from colleagues and subordinates. Fearless disregard of public opinion or criticism, in advocating or executing measures which, in his judgment, were commendable and right; independence of thought, and great originality in conception, design, and methods; untiring energy and industry. His active mind, dominating a somewhat frail physique, seemed never to be at rest.

At his home, there was ever a rare and genial hospitality and an unfailing and active interest in the welfare of the people among whom he lived.

Major Lewis was gifted with correct taste and appreciation of works of art, and was a liberal subscriber to funds for the promotion thereof, as well as to other measures of public character. His private charities are known to have been large, but were ever carefully concealed.

Briefly, Major Lewis was public spirited, self-reliant, masterful, fearless, outspoken, faithful in duty, talented, successful, charitable, and honorable in all his dealings.

Like all men of strong character, he was not faultless, nor without vigorous opponents and some personal enemies; and, yet, by one and all, his death was felt to be a calamity; and his imperative dictation, that there should be "no flowers, no funeral, no fuss" when he should pass away, could not prevent the large attendance of eminent men, long-time associates, and warm-hearted friends, from near and far,

at the brief but deeply impressive service, when, at the twilight hour, his remains were placed in the family vault at Mt. Olivet.

Then, too, as the twilight faded, for 300 sec. all traffic ceased, no wheels were in motion, on the entire system of the Nashville, Chattanooga, and St. Louis Railway, and on the street railways of the City of Nashville.

Major Lewis was elected a Member of the American Society of Civil Engineers, on March 5th, 1873, and served as a Director in 1903-05 and 1912-14. He valued his membership in the Society very highly, and, until his death, maintained an active interest in its affairs.

ALEXANDER WILLIAM MacCALLUM, M. Am. Soc. C. E.*

DIED FEBRUARY 3D, 1917.

Alexander William MacCallum was born in Edinburgh, Scotland, on July 25th, 1859.

He served an apprenticeship of four years to a mechanical trade at the shops of the Central Railroad of New Jersey, at Ashley, Pa. This apprenticeship was finished in 1881, and he then entered Lafayette College, at Easton, Pa., from which he was graduated in June, 1885, with the degrees of C. E. and M. E.

During 1886, and a part of 1887, Mr. MacCallum was employed as Engineer and Assistant Superintendent at the Massillon Engine Works.

In 1887 he was engaged by Samuel R. Bullock and Company, of New York City, as Engineer and Superintendent of the water-works at Massillon, Circleville, and Defiance, Ohio, thus entering the field of water-works engineering, in which he afterward specialized.

In March, 1889, Mr. MacCallum was appointed Engineer and General Manager in charge of all the public utilities of R. D. Wood and Company. While in this position, he supervised the building and rebuilding of many of that Company's water-works plants, as well as the management of several of its gas and light plants, including those at Denison, Tex.; Mobile, Florence, and Tuscaloosa, Ala.; Pensacola, Tampa, and Lake City, Fla.; Macon, Ga.; Vincennes and Greencastle, Ind.; Sharon, Corry, Chester, and Marcus Hook, Pa.; Owego, N. Y.; and Millville, N. J.

On numerous occasions, he acted as Appraiser and Arbitrator for various companies, notably for the Trinidad, Colo., Water-Works (1896); the Macon, Ga., Gas Light and Water Company (1910); the Warren, Pa., Water-Works (1911); the Ashland, Ky., Water Company (1911); as well as the Kane, Pa., Water Company, the Catlettsburg, Kenova, and Cereda, W. Va., Water-Works, etc.

From March, 1906, until his death, Mr. MacCallum was Consulting Engineer and General Manager, in charge of a number of public utilities for financial interests, with headquarters at Philadelphia, Pa., devoting his spare time to valuations and appraisals and to examinations and reports, for bankers and private investors, on water-works properties in various States. He was probably best known as the Secretary and General Manager of the New Chester Water Company, of Chester, Pa., and also of the Linwood, South Chester, Ridley,

* Memoir prepared by the Secretary from information furnished by George W. Fuller, M. Am. Soc. C. E., Robert C. Wheeler, Assoc. M. Am. Soc. C. E., and Harry S. Hopper, Esq.

Chichester, Edgemont, and Delaware Water Companies, and of the Greencastle and Vincennes, Ind., Water Corporations.

On February 3d, 1917, while apparently in the best of health, Mr. MacCallum suffered a stroke of apoplexy which caused his death.

On October 13th, 1887, during his residence in Massillon, Ohio, Mr. MacCallum was married to Grace F. Weaver, of Cleveland, Ohio. He is survived by his wife and five children, Mrs. Nellie Williams, Jane Gordon, Grace Elizabeth, Walter Wood, and Alexander William MacCallum, Jr.

His most striking characteristic was his cheerfulness and optimism and his devotion to his work, his family, and his immediate friends, among whom his loss will be keenly felt. Although exceedingly modest and retiring, he was withal a shrewd business man. He used sound judgment in the application of his engineering knowledge in the water-works field, and won the confidence alike of citizens and public officials, as well as of his associates, in the management of the various water-works properties. Under his quiet manner, however, Mr. MacCallum had great strength of character, devotion of purpose, and deep religious convictions.

He had made his home at Merchantville, N. J., for many years, and always took an active interest in local public affairs. Among other things, he served for a long time as a member of the Sewerage Commission which inaugurated and constructed the improved sewerage system for the town.

At a meeting of the Executive Committee of the New Chester Water Company, on February 21st, 1917, Mr. Harry S. Hopper, the Treasurer of the Committee, who was also an intimate friend of Mr. MacCallum, made the following statements:

"It is with a feeling of deep sadness that I report the sudden death on February 3d, 1917, of our late Secretary and General Manager, Mr. Alexander W. MacCallum.

"The loss of Mr. MacCallum seems irreparable to me personally, and also to the Water Works he managed, as well as the Water Works community generally. He was connected with our interests for nearly 30 years, and was much more than the title given him indicates. He took a deep personal interest in each plant and could not have been more faithful and painstaking had he owned the properties. He always put the 'ladies' first, meaning Mrs. Little, Mrs. Lippincott, and Mrs. Hopper, and never spared himself where their interests were involved.

"He built the properties up in the last ten years to a point of efficiency and profit toward which he had been aiming, and looked forward to the completion of the work planned at Chester as the crowning effort of his life; it is difficult to understand why he should not have been allowed to realize his desire."

The following resolutions were also unanimously adopted at the meeting:

"*Resolved,* That, in the death of Mr. MacCallum the Company has met a loss that cannot be supplied. Mr. MacCallum's character included an extraordinary combination of qualities. To a thorough technical knowledge of the scientific branches of the business, which he had acquired as engineer, he united the broad knowledge of business, which he had earned by conscientious devotion to work, and he had also the higher qualities which the experience of responsibility cheerfully and successfully borne brings to all. His sympathy with human nature was deep and evident, and his qualities of friendship were sincere and enduring.

"*Resolved,* That the Board desires to extend its deepest sympathy to Mrs. MacCallum and her family, and directs that a copy of these resolutions be forwarded to her."

Mr. MacCallum was elected a Member of the American Society of Civil Engineers on September 2d, 1914.

DANIEL McCOOL, M. Am. Soc. C. E.*

DIED NOVEMBER 30TH, 1916.

Daniel McCool was born at London, Ont., Canada, on January 9th, 1850. He was educated at the Jesuit College in Quebec, and, later, was graduated from the English High School in that city.

In 1869, Mr. McCool moved to Niagara Falls, N. Y., and later to Auburn, N. Y., and his first engineering experience was gained as Rodman on the fortifications at Point Lewis where he was employed for two years.

He then entered the employ of the New York and Oswego Midland Railroad Company as Assistant Engineer on construction. In 1873, he became Assistant Resident Engineer on the New York Central and Hudson River Railroad, in which position he had charge of the four-track improvement west of Syracuse, N. Y., under the Chief Engineer. During the last five years of his service with this Company, Mr. McCool, in addition to his engineering duties, served as Assistant Superintendent in charge of traffic.

In 1880, Mr. McCool was appointed Private Secretary to the General Manager of the Michigan Central Railroad and also had charge of the Engineering Department of the Company. While in this position, he designed and constructed the new freight yards and stockyards at Detroit and made the plans for the new Passenger Station at that place, as well as the improvements connected therewith.

In 1882, he entered the service of the Detroit, Mackinaw and Marquette Railroad Company as General Superintendent and Chief Engineer. In this capacity he had charge of the car ferry service over the Straits of Mackinac, and introduced the first successful ice-crushing craft, by placing a propeller at the bow as well as at the stern of the ferry. While in the employ of this Company he extended the line to Ishpeming and Negaunee, Mich., in order to tap the extensive iron mines in that region.

In 1885, Mr. McCool was made General Manager of the St. Joseph and Grand Island Railroad, with headquarters at St. Joseph, Mo. He was also President of the St. Joseph Terminal Company, and built the Kansas City and Omaha Railroad, a distance of 200 miles, as a feeder to the St. Joseph and Grand Island.

In 1888, he became General Manager of the Santa Fé and California Railroad, and during his connection with that Company added 100 miles to its lines.

* Memoir prepared by the Secretary from information on file at the Society Headquarters.

In 1889, Mr. McCool retired from railroad work and made an extended tour in Europe. On his return to the United States, he organized, in 1898, the Newaygo Portland Cement Company, at Newaygo, Mich., with which he was identified at the time of his death, having served as its President for many years and having been largely instrumental in putting it on a paying basis. Under his direction, the marl beds near Newaygo, belonging to the Company, were developed, the local power dam was built, and the great plant of the Company was constructed. In addition to his work as President of the Newaygo Portland Cement Company, Mr. McCool was prominently identified with the Edison Electric Light Company, of Grand Rapids, Mich., of which he became President in 1901. He was also interested in many other local enterprises.

In the summer of 1899, Mr. McCool sustained serious injuries in a runaway accident from which he never fully recovered. A short time before his death, he had entered the Henry Ford Hospital, at Detroit, Mich., for treatment, and had returned to his home in Grand Rapids apparently greatly improved. He had continued actively in the business of the Newaygo Portland Cement Company and had visited the plant only a few days before the brief illness which caused his death.

Mr. McCool is survived by his widow, who was Miss Kate Fisher, of Batavia, N. Y., and one brother, Mr. William A. Tench, of Detroit, Mich.

In Grand Rapids and Newaygo, Mr. McCool was widely known for his benevolent and philanthropic work, having been actively identified with many charitable movements in both places. In the business world his influence was strong and progressive, and he commanded high esteem among men of large affairs. His funeral took place at Batavia, N. Y., and out of respect for him who had been its founder and President, the hour was marked by the complete cessation of operations at the plant and power house of the Newaygo Portland Cement Company.

Mr. McCool was elected a Member of the American Society of Civil Engineers on September 5th, 1883.

THEODORE HALL McKENZIE, M. Am. Soc. C. E.*

DIED MAY 3D, 1916.

Theodore Hall McKenzie was born in Yalesville, Conn., on March 29th, 1847. His father, William McKenzie, was a native of Scotland, who came to America when a young man and engaged as a contractor in the construction of buildings and stone bridges. His mother, Temperance (Hall) McKenzie, was a descendant of an old Connecticut family, and had great strength of character.

During boyhood, Theodore Hall McKenzie attended schools in Wallingford and Meriden, Conn., and, later, took the Scientific Course at the Literary Institute, in Suffield, where he studied Surveying. When still a lad, he read many scientific books, and during 1866-68 he assisted his father in the construction of public works.

From 1868 to 1872 he worked with location and construction parties on several railroads in Connecticut and Massachusetts. During this period, he took private lessons under professors of the Sheffield Scientific School of Yale. He then became, for two years, Division and Resident Engineer on the Providence and Springfield Railroad. From 1875 to 1878, he was City Engineer of Meriden, Conn., where he prepared plans, under the direction of the late E. S. Chesbrough, Past-President, Am. Soc. C. E., for the sewerage of that city, and also built reservoirs for an addition to its water supply.

In 1878 Mr. McKenzie was elected Secretary of the Peck, Stow and Wilcox Company, hardware manufacturers, of Southington, Conn., which position he held for 10 years. During this period, he designed and built the water-works of Southington and Plainville. He also planned a sewage disposal plant for Meriden, which was the first in Connecticut. A paper on "The Water-Works of Southington, Connecticut",† was written by him and presented before the Society.

After 1888, Mr. McKenzie maintained offices in Southington and Hartford, and served as Consulting Engineer on the construction of fourteen water-works and twelve sewerage systems. Among these may be mentioned the water-works at Naugatuck, Litchfield, Wallingford, Simsbury, Terryville, Newton, and South Manchester, Conn., and Brewster, N. Y.; sewage disposal plants at Manchester, South Manchester, Norfolk, Bristol, Ridgefield, and Sharon, Conn., Johnstown and Gloversville, N. Y., and Princeton, N. J.; also water-power developments at Berlin, Conn., and Croton Falls, N. Y. Since 1906, he had been largely engaged in the appraisal of mills and water power on the Croton and Ashokan water-sheds for the supply of the City of New

* Memoir prepared by Mansfield Merriman, M. Am. Soc. C. E.

† *Transactions*, Am. Soc. C. E., Vol. XV (1886), p. 885.

York, and on similar work for the Barge Canal in New York State. He was frequently an expert witness in litigations regarding such property.

In 1887, the Legislature of Connecticut constituted a State Board of Civil Engineers to inspect dams and to approve plans for new dams and reservoirs; Mr. McKenzie was a member of this Board for more than a quarter of a century, and, during part of that time, he was its Chairman.

Mr. McKenzie was Secretary and Superintendent of the Southington Water Company for 27 years, and Secretary and Manager of the Terryville Water Company for 4 years. He served as the Engineer Member of the Connecticut State Board of Health for 20 years. He was a member of the Connecticut Society of Civil Engineers, of the New England Water-Works Association, and of the American Public Health Association. He was also a member of the Masonic fraternity, and of the Baptist Church.

He was a hard worker in his profession, but, in his home, he loved the recreation of music, and out-of-doors he regarded horses as one of the best relaxations from the cares of life. His favorite counsel to young men was to learn thoroughly some business or profession and to abstain entirely from tobacco and liquors. Mr. McKenzie was held in esteem by his business associates and by the community in which he lived on account of his high character and pleasing personality.

He died suddenly, from an attack of heart disease, in his office in Southington, shortly after having arrived there to begin the work of the morning.

Albert B. Hill, M. Am. Soc. C. E., of New Haven, Conn., writes as follows regarding Mr. McKenzie:

"I was not intimately acquainted with him and was never associated with him on any work, although I was on the opposite side in several water cases. His services were much in demand as an expert in hydraulic cases, not only in his own State but in neighboring States. He was very zealous for the interests of his clients and considerate of his opponents.

"He had a very pleasant, agreeable personality, and will be greatly missed in New England engineering circles."

Robert E. Horton, M. Am. Soc. C. E., of Albany, N. Y., writes as follows:

"I knew Theodore H. McKenzie quite intimately during the period from 1906 until the time of his death. My association with him was largely in the matter of preparation of technical evidence in water-power claims on Esopus Creek. However, he was also associated with me in the defense of certain water-power and flood claims arising from the construction of the Barge Canal, we being both employed by the State of New York on these matters. We were also associated on certain matters involving water-power and water supplies, quite a number

of matters, in fact, in New England. I would say of Mr. McKenzie that one might on first acquaintance easily misjudge him because of his inclination to be apparently brusque and very direct and frank in anything he had to say. The frankness and directness were true characteristics of the man. The brusqueness was apparent, not real, as on better acquaintance he was found to be exceedingly genial and cordial, in fact, I have known but few men who were so carefully and consistently loyal and devoted to their friends as Mr. McKenzie.

"In professional work Mr. McKenzie may be said to have belonged to the older school of practical engineers. The methods which he adopted for the solution of problems were nearly always simple and direct. He had had a very wide experience in engineering work in certain lines, especially water supply, sewerage, and water-power work. This experience had given him most excellent judgment in matters to which it related. He would reach conclusions at times in so simple and direct a manner as perhaps to arouse suspicion on the part of some of the younger generation who believe that correct results in similar matters can only be reached through long processes of calculation. To one, however, who knew Mr. McKenzie's rich experience, his judgment would be most highly esteemed.

"Mr. McKenzie was at times somewhat severe in his criticism of those whom he believed to be dishonest or unprincipled in their actions. He was himself apparently incapable of anything but absolute frankness. This trait was often evident in his work as an expert witness, wherein he would very readily admit any necessary qualification of testimony he had given which was brought to his attention. This was at times perhaps a little distressing to attorneys with whom he was associated, but, on the other hand, he numbered among his most steadfast friends many attorneys prominent in the trial of technical cases, both those with whom he had been associated and those whom he had opposed."

On October 11th, 1871, Mr. McKenzie was married to Mary E. Neal, daughter of Roswell A. and Eunice (Atkins) Neal, of Southington, Conn. She survives him, with two sons, Samuel H. and William A. McKenzie, and two daughters, Eunice J. and Fanny L. McKenzie.

Mr. McKenzie was elected a Member of the American Society of Civil Engineers on September 7th, 1881.

STANLEY ALFRED MILLER, M. Am. Soc. C. E.*

DIED MAY 13TH, 1917.

His many friends and associates were shocked beyond measure by the dreadful tragedy which culminated in the death of Stanley Alfred Miller and his assistant, Mr. Epaphroditus Hawkins, which took place about 42 miles from La Romana, Santo Domingo, on the morning of May 13th, 1917.

Stanley Alfred Miller was born in New Orleans, La., on May 27th, 1882, of one of the fine old families which have made his native State so famous by their ideals and accomplishments, and whose traditions he consistently maintained. He was educated at local schools and at the Louisiana State University, where he was one of the youngest students.

His professional work began like that of most engineers, as Rodman, Levelman, Instrumentman, etc. Then, he was Assistant Engineer on sewers, and on levee construction and railways. At the age of 19, however, Mr. Miller was Assistant Engineer to the Board of Public Works of Mobile, Ala., engaged in design and construction. As Assistant to various consulting engineers he was employed on the survey, design, and construction of sewers and water-works for Baton Rouge, La., Mobile, Ala., Dallas, Tex., and Ardmore and South McAlester, Okla. In 1904, he served as Resident Engineer on the Chihuahua and Pacific Railroad, and, in 1905, he was with S. Pearson and Son, Limited, Contractors, of London, England, on the construction of Port Works at Coatzacoalcos (now Puerto, Mexico), having been for a time in direct charge of the contractors' labor forces on the terminal yards and jetties. This will be recognized by all engineers as a position of great responsibility for a man of his age.

This was followed by his work for the Mexican Light and Power Company, near Necaxa, Mexico, in 1906, in local charge of the construction of Earth Dams Nos. 1 and 3, and Tunnel No. 1, where his infinite care and attention to every detail proved to be the source of much valuable information to his associates.

While living in the semi-permanent camp near the dams, Mr. Miller was returning to his quarters after a midnight inspection of some of the work, when he stumbled over a severely wounded Mexican laborer. Having no assistance, and in spite of the fact that he was not a large man physically, he picked up the wounded man, carried him to his own cabin, and telephoned for a doctor. It was raining heavily, and the doctor was miles away attending to other patients, so Mr. Miller, by the light of a flickering and temporary arc lamp, dressed the abdom-

* Memoir prepared by Verne LeRoy Havens, M. Am. Soc. C. E.

inal wounds, the operation requiring many stitches. The man was doomed in any event, but Mr. Miller's efforts maintained the spark of life until his family could get to him.

Mr. Miller was engaged in the construction of a 7-mile sewer for Paducah, Ky., and paving and sewerage for Cairo, Ill., in 1906 and 1907. He was also Assistant Engineer on hydro-electric projects in California and Colorado in 1907 and 1908.

During the next three years, he was in charge of the Azua Irrigation Project, in Santo Domingo. On this work his duties were manifold and trying, but he completed them to his own satisfaction, and to those who knew him, the meaning of that statement will be clear, as he was his own most meticulous taskmaster. His work consisted of surveys and plans for, and reports on, the watering of 50 000 acres of sugar lands, and a hydro-electric development. After this, he took charge of the location of 55 miles of improved highways for Copiah County, Miss., and the construction of a portion of the work. He then joined the forces of the Uruguay Railway Company, and was placed in charge of railway location and construction, and the building of a reinforced concrete wharf, bridges, buildings, etc.

When visiting one of the camps under his supervision on this work, at Paloma del Sarandi, Mr. Miller found that one of the engineers had died in such a manner that local burial was impossible, and, in view of the refusal of the members of the party to assist him, he himself prepared the body for transportation. Then, with no food from the morning of one day until darkness of the second day, he traveled 60 miles, continuously, through the rainy night, in an ox-cart, secured help at the county seat, bought the ground, obtained authority for the burial, and organized a funeral, in order that the engineer might have the last rites due him, and that his parents, in far off Europe, might know where the body of their son lay.

On two separate occasions, Mr. Miller insisted on accompanying the writer on long trips without explaining his reasons for doing so, but the writer learned months afterwards that he had heard of threats against him for fancied errors in company policy.

From October, 1914, until December, 1916, Mr. Miller was engaged in general consultation practice at his home in Paducah, Ky. In this capacity, he designed additional sewers for Paducah, reported on drainage projects in Kentucky, a hydro-electric plant in Mississippi, and rates for a public utility corporation. He also became well known throughout his adopted State by his earnest efforts, through the press and various organizations, to improve the status of the engineer, to eliminate engineering work from politics, and to further the general interest in good roads. He had built up a good practice, his work being continued during his last absence in Santo Domingo.

On December 9th, 1916, Mr. Miller sailed from New York for Central Romana, a sugar estate of the South Porto Rico Sugar Company, to "study a couple of rivers for hydro-electric and irrigation development", and lived in camp to push the work. This apparently was progressing nicely until the fateful Sunday morning of May 13th, 1917, when his camp was attacked shortly after daylight by a bandit with more than 100 men. The little party was quickly overpowered and led to their death, which took place about an hour afterward. One Spanish helper and a British negro were tied near Mr. Miller and Mr. Hawkins, but only the latter were executed. Mr. Miller explained to the bandit the nature of his work and its non-military character, but was told that "their fault was being Americans." His only plea, that he be allowed to write to his wife, was denied him. Mr. Miller's religious convictions were known only to those who knew him intimately, and it was his marvellous faith that the Divine Will would be done, and his maxim that "one must meet his portion without flinching", which undoubtedly formed the basis for his most extraordinary courage in facing death.

To those who stay behind, the loss is always keen and enduring, but the loss of Mr. Miller is felt with unusual regret. His rapid progress is proof to all of his professional value, and the younger engineers already counted him as a leader. His State will feel the loss of an able, analytical citizen, whose spirit was of the metal that tempers entire communities. The writer was associated with Mr. Miller on various works, and knows how much his friends will miss his counsel, and how much all those who knew him will miss the fine innate sense of justice, coupled with a knowledge of human weaknesses, which made his quietly expressed opinions grow on one like the conception of a fundamental law.

Mr. Miller was married to Miss Ann Bradshaw, of Paducah, Ky., on March 16th, 1911. Their first daughter, Miss Vauban Miller, died and was buried at sea, but their second daughter, Stanley Ann Miller, who was born just six weeks before her father went to Santo Domingo, together with her mother, survives him. He is also survived by his father and mother, Mr. and Mrs. W. R. Miller, his brother, Mr. W. R. Miller, Jr., and his sister, Mrs. W. Molton Evans, all of Baton Rouge, La.

Mr. Miller was a member of New England Water Works Association, the Boston Society of Civil Engineers, and the Sigma Alpha Epsilon Fraternity. He was elected a Junior of the American Society of Civil Engineers on February 4th, 1902, an Associate Member on April 6th, 1909, and a Member on June 24th, 1916.

WILLIAM RIDLEY NEELY, M. Am. Soc. C. E.*

DIED APRIL 11TH, 1916.

William Ridley Neely, the son of Robert Johnson Neely and Elizabeth Norfleet Ridley, was born on August 20th, 1872, in Portsmouth, Va. He was educated at private schools, first at a "Dame's school" in Portsmouth, and then at Norfolk Academy, of Norfolk, Va. He also attended the University of Virginia from 1888 to 1890.

The early years of Robert Johnson Neely, his father, were spent at Newton, Pa., where the old Neely family homestead, built before the Revolution, is now the property of the Daughters of the American Revolution. Early in life he went to Portsmouth, Va., where he was engaged as a lumber manufacturer. He became so identified with the South that he fought with the Confederate Army all through the Civil War. Mr. Neely's mother, Elizabeth Norfleet Ridley, is a collateral descendant of Bishop Ridley who was burned to death for religion's sake in the time of Mary Tudor. She was born and brought up on a plantation near Portsmouth, Va., and was educated in convents in South Carolina and at Richmond, Va. Her father was a well-known landed proprietor of his section, a very fine type of the ante-bellum country gentleman. William Ridley Neely's parentage and traditions were all of the South, and he always took great pride in the development of his home State.

Mr. Neely entered the service of the United States Engineer Department, in the Vicksburg District, in October, 1891, and remained in the employ of the Government for 8 years, being engaged mainly on soundings, precise level work, topographical surveys, and computations, along the Mississippi and tributary rivers. In March, 1899, he was furloughed to take service with the Isthmian Canal surveys in Nicaragua, and remained there until April, 1900, at which time he returned to the Vicksburg District and remained until 1904, in charge of test borings and drafting plans and designing locks, dams, and other structures. In connection with the test borings in the bed of the Ouachita River, several deep borings were made to determine the geological structure of the valley, notably at Catahoula Shoals, Columbia, Rock Row Shoals, Jacks Island, and Newport Landing. The information obtained has been utilized in the later reports on the geology of Louisiana. The plans for the Ouachita locks and dams embraced the designing of foundations in treacherous material, concrete walls subject to earth and water pressure, steel lock gates, and a variety of operating machinery, as well as the solution of many hydraulic problems.

* Memoir prepared by James F. Sanborn, Assoc. M. Am. Soc. C. E.

In May, 1904, Mr. Neely was furloughed to take up engineering work at the West Point Military Academy, where he remained until October, 1906, on work connected with the increase of the water supply for the military reservation there. This involved the location of the pipe line through most difficult country and its actual laying, the length of the line being approximately 6 miles, and the pipe 20 in. in diameter. Mr. Neely did most of the field work connected with this project, and his superior officer considered the service which he rendered very satisfactory. He also assisted in designing the filters which were to be built at West Point.

In October, 1906, Mr. Neely entered the service of the State of New York, as Assistant Engineer in the Engineering Department of the Barge Canal, and continued with that work for more than two years. D. A. Watt, M. Am. Soc. C. E., who was at that time Assistant Engineer, in charge of the Designing Office of the New York State Barge Canal, says:

"He [Mr. Neely] assisted me during that time in preparing the plans for some of the chief sections of the canal, such as the cross-country line just west of Rochester, which comprised two high-lift locks, as well as very heavy cuts and embankments; the line through Phœnix on the Oswego River; and the line through Little Falls. The latter included a lock of 40½ ft. lift, as well as difficult work in carrying the canal through a narrow gorge and along the side of a rock bluff overhanging the Mohawk River. I found Mr. Neely an able man. He was a very hard worker and had originality and accuracy, and I was very sorry to lose his services. He left to accept a more promising position with the State Water Supply Commission."

In the spring of 1908, Mr. Neely was employed as Resident Engineer in charge of two field parties making topographical surveys and maps of the basin of the proposed big Sacandaga Storage Reservoir, near Northville, N. Y., as well as surveys of the storage possibilities at Piseco, Pleasant, and Schroon Lakes. In 1909, he was re-engaged by the State Water Supply Commission, but soon after resigned. Horace Ropes, M. Am. Soc. C. E., speaks of him, as follows: "Mr. Neely was an active and industrious worker, an agreeable person, well liked by those associated with him."

From June, 1909, to April, 1913, Mr. Neely was employed by the Board of Water Supply of the City of New York on the construction of Section 1, Wallkill Division, of the Catskill Aqueduct, at New Paltz, N. Y., of which section he had charge. The work consisted of a standard type concrete aqueduct, in cut and cover, along a strip on the east slope of the Shawangunk Mountains, mainly in rock cut, and a grade tunnel driven toward the north under Bonticou Crag to meet the tunnel driven from the other side of the range. In contrast with the soft-ground problems of his early experience along the lower

Mississippi River, Mr. Neely greatly enjoyed the work and methods used for the hard-rock excavation, timbering, and concrete construction in the tunnel, and the difficulties encountered in open-cut work. In connection with his duties of supervising the construction, he worked out several problems of design for special structures and unusual conditions, and also had the gratification of executing this work. He was a member of the Catskill Aqueduct Association and took great interest in his relations with the engineers he met on that work.

On April 10th, 1913, Mr. Neely resigned to enter the contracting field. He built several miles of the special concrete post and wire fences along the Aqueduct, which occupied him until the spring of 1915. At the time of his death on April 11th, 1916, he was living in Portsmouth, Va.

He was married on February 20th, 1901, to Annie Calvert Jones, of Vicksburg, Miss.

Mr. Neely was a member of the General Alumni Society of the University of Virginia. He was elected a Member of the American Society of Civil Engineers on July 9th, 1906.

HAROLD PARKER, M. Am. Soc. C. E.*

DIED NOVEMBER 29TH, 1916.

Harold Parker, the son of George Alanson and Harriet Newhall Felton Parker, was born in Charlestown, Mass., on June 17th, 1854. He was a direct descendant of Captain Joseph Parker, who settled in Newbury in 1638, and of Nathaniel Felton, who settled in Salem in 1633.

After receiving a preliminary common school education in Philadelphia, Pa., and Lancaster, Mass., Mr. Parker entered Phillips Exeter Academy, from which he was graduated. He then entered Harvard College, where he continued his studies during 1871 and 1872.

Leaving college for a business career, he was first employed in the capacity of Foreman with the Pennsylvania Steel Company. In 1874, he returned to his home town, Lancaster, Mass., and became a member of the civil engineering firm of Parker and Bateman, continuing as such until his death.

Notwithstanding the fact that he was of modest disposition and not of the office-seeking type, Mr. Parker was prominent in the political affairs of Lancaster, and was twice elected to represent his district in the State Legislature. His best years in public service, however, were devoted to the work of building up the highway system of his native State. In 1900 he was appointed by the Governor as a member of the Massachusetts Highway Commission, and on the resignation of William E. McClintock, M. Am. Soc. C. E., in June, 1908, he was made Chairman, which position he held until 1911, when he resigned again to devote his time to other than public office. His ability and tireless industry added to the good repute for gaining progress and results for which this Commission had already become widely known. The revolution in methods of highway transportation and consequent revolution in methods of highway construction was at its height during Mr. Parker's administration as Chairman of the Commission, and many who are continuing the work so ably carried on by him, are indebted to him for his thought and foresight.

Immediately following his resignation as Highway Commissioner, he became Vice-President of a large contracting firm, thus continuing to devote his time to road and pavement work.

In 1913 and 1914, Mr. Parker served the State of New York in an advisory capacity, being designated by Governor Dix as a member of the Advisory Board of the Highway Department.

Being a lover of nature, he spent his leisure in the fields and woods, taking much interest in the forests and natural resources of

* Memoir prepared by A. W. Dean, M. Am. Soc. C. E.

his neighborhood and State, and considering as a pleasure rather than a duty his services as Chairman of the Massachusetts Forestry Commission and of the Wachusett Mountain Reservation Commission.

Indication of Mr. Parker's recognition by others in his chosen field of effort is shown by his selection as President of the Massachusetts Highway Association and of the American Road Builders' Association, in both of which societies he took great interest.

Mr. Parker was a man of the sturdy New England type, of high principles, and a most true friend to those who won his friendship.

He is survived by his widow, Elizabeth B. (Bartol) Parker (to whom he was married in 1884), and by one son and two daughters.

Mr. Parker was elected a Member of the American Society of Civil Engineers on June 7th, 1899.

WILLIAM RODNEY PATTERSON, M. Am. Soc. C. E.*

DIED JULY 19TH, 1916.

William Rodney Patterson was born at Effingham, N. H., on November 4th, 1854.

He was graduated from Dartmouth College in 1876, and, within a few months, was associated with the late Enos M. Barton, of the Western Electric Company. For many years, Mr. Patterson was the Plant Engineer of the Western Electric Company, and had direct charge of the construction and maintenance of all its many buildings, not only in the United States, but also of its various plants in Europe and the Orient.

He was directly responsible for the invention and perfection of the dry-core telephone cable now so generally used throughout the civilized world, as well as of the process of extruding a lead sheath over the cable. He took out more than 100 patents in connection with the development of the telephone cable.

In 1911, Mr. Patterson retired from the Western Electric Company and, until his death, which occurred on July 19th, 1916, after an illness of some months, he was senior member of the firm of Patterson and Davidson, Consulting and Designing Engineers, in Chicago, Ill.

He was a man of quiet reserve, but was nevertheless endowed with an interesting and genial personality, and any one who came in contact with him, in business or socially, became at once his stanch friend. By nature a man of the simplest tastes, he was a great reader and a keen observer. His hobby, during his spare moments, was the preparation of colored lantern slides of architectural and engineering objects, and his library contains many thousands of such slides colored by his own hands. He was a great traveler, and, during the latter years of his life, spent at least one-half his time on foreign shores.

As an executive, Mr. Patterson had few superiors. He had the peculiar natural gift of being able to select his assistants and of having the wise judgment of transferring to them authority and holding them responsible for results, he himself not devoting his valuable time to minute details of work or organization.

While in the employ of the Western Electric Company, he was recognized as its construction genius. He had seen the Company develop from about 25 employees until, at the time of his retirement,

* Memoir prepared by F. E. Davidson, M. Am. Soc. C. E.

they numbered more than 50 000; and in that time the yearly sales had increased from a few thousand to about eighty million dollars.

Engineers will remember Mr. Patterson in connection with the development of the telephone industry, as without doubt its rapid development has been due more directly to the invention and perfection of the dry-core telephone cable than to any other one thing.

Mr. Patterson was elected a Member of the American Society of Civil Engineers on May 4th, 1909.

JOSEPH RAMSEY, Jr., M. Am. Soc. C. E.*

DIED JULY 7TH, 1916.

Joseph Ramsey, Jr., the son of Joseph and Mary (Patterson) Ramsey, was born at Pittsburgh, Pa., on April 17th, 1850. He was educated in the public schools of Pittsburgh, and completed his studies at the Western University of Pennsylvania in 1869.

Mr. Ramsey began his railroad career as a Rodman in the Engineer Corps of the Pittsburgh, Cincinnati and St. Louis Railroad. In 1870 he was made Engineer in charge of the construction of the Dresden Cut-off, at Dresden, Ohio, but resigned to accept the position of Assistant Engineer on location and construction of the Bell's Gap Railroad, and, a year later, was made Assistant Engineer of the Lewiston Division of the Pennsylvania Railroad.

Returning to the Bell's Gap Railroad, in 1872, Mr. Ramsey was made Chief Engineer, and after its completion, in 1873, its Superintendent, in which position he continued until January, 1879, when he entered the service of the Pittsburgh, New Castle and Lake Erie Railroad as Chief Engineer and General Superintendent. He resigned this position in September of that year, to take a similar one with the Pittsburgh Southern Railroad. Mr. Ramsey's unusual abilities were speedily recognized, and he was offered the position of General Manager of the Pittsburgh, Chartiers and Youghiogheny Railroad. While thus engaged, he became General Manager of two other branch roads and the Chartiers Block Coal Company.

From 1883 to 1890 Mr. Ramsey served as Chief Engineer of the Cincinnati, Hamilton and Dayton Railroad, resigning to become Assistant to the President of the Cleveland, Cincinnati, Chicago and St. Louis Railroad, which post he retained until 1892. From 1890 to 1895 he was President of the Peoria and Pekin Union Railway and, from 1891 to 1893, Vice-President of the Cincinnati, Wabash and Michigan Railroad, having charge of the Operating and Traffic Departments of both these companies, while holding the Presidency of the Findlay Belt Railway. In June, 1891, he was made General Manager of the Cleveland, Cincinnati, Chicago and St. Louis Railway, and, in 1892, was elected General Manager and Vice-President of the Dayton Union Railroad. In April, 1893, Mr. Ramsey accepted the position of General Manager of the Terminal Railroad Association of St. Louis, which position he held until December 1st, 1895, when he was made Vice-President and General Manager of the Wabash Railroad Company. He was made President of the entire Wabash System in 1901, a position which he filled for five years with signal success.

* Memoir prepared by John P. Ramsey, Esq.

Mr. Ramsey was also President of the Wheeling and Lake Erie Railroad and the Wabash-Pittsburgh Terminal Railroad from 1901 to 1905; the Ann Arbor Railroad from 1902 to 1906; and the Western Maryland Railroad and West Virginia Central Railroad from 1903 to 1905. He also served as President of the Pittsburgh and Chicago Railroad, the Lorain and Ashland, and the Ashland and Western Railways. In 1913, Mr. Ramsey became President and General Manager of the Lorain, Ashland and Southern Railroad, which he had built. While President of the Wabash Railroad he was sent to the Paris Exposition of 1900 as Railroad Commissioner from the United States.

Mr. Ramsey was a Director of the Louisiana Purchase Exposition, at St. Louis, Mo., and Vice-President of its Committees on Electricity and Electrical Appliances and Transportation. He was also connected with several companies as Director and Committeeman. He was a member of the Commercial and Noonday Clubs, of St. Louis, Mo., the Duquesne Club, of Pittsburgh, Pa., and the Toledo Club, of Toledo, Ohio.

Mr. Ramsey was married on April 8th, 1873, to Laura, daughter of James E. Palmer, of Zanesville, Ohio, who, with two sons, James Palmer and John Patterson, and three daughters, Mrs. Helen R. Fowler, Miss Jane M. Ramsey, and Mrs. Mary R. McIntyre, survives him. His death occurred at East Orange, N. J., on July 7th, 1916.

Mr. Ramsey was elected a Member of the American Society of Civil Engineers on May 1st, 1889, and served as a Director from 1900 to 1902, inclusive.

THOMAS FRANKLIN RICHARDSON, M. Am. Soc. C. E.*

DIED DECEMBER 26TH, 1915.

Thomas Franklin Richardson was born at Woburn, Mass., on February 10th, 1855. He was of Colonial ancestry, having been descended from Samuel Richardson who came to this country about 1630 from the south of England and settled in what is now Winchester, Mass.

Mr. Richardson was graduated in 1872 from the Boston English High School, and began his career as a Civil Engineer in May, 1873, as Rodman on the location and construction of the aqueduct for the Sudbury Water-Works, for the City of Boston. When the work was finished in December, 1878, he had been promoted to the position of Instrumentman.

From December, 1878, to October, 1879, he was employed as Assistant Engineer for the City of Chelsea, Mass., in charge of the construction of the main trunk sewer for that place.

In January, 1880, Mr. Richardson went West as Topographer on surveys for the location of the Atlantic and Pacific Railroad (now part of the Santa Fé System), and in seven months was made Division Engineer, which position he held until June, 1882. As Division Engineer, he had charge of the construction of about 25 miles of railroad in New Mexico and Arizona and was also active in the management of that portion of the road. His work here included the construction of the Cañon Diablo and the Padre Cañon Bridges in Arizona, the former of which was a notable piece of engineering for that time, being a steel trestle about 225 ft. high and 600 ft. long. Labor conditions were extremely bad in this country, and water for construction purposes had to be hauled from 25 to 30 miles, largely by teams, thus making the work very difficult.

In October, 1882, Mr. Richardson joined the Engineering Corps of the Mexican Central Railway Company as Assistant Engineer. In this capacity he had charge of the construction of the Encarnacion Bridge, 175 ft. high and 1 300 ft. long. At that time this bridge was considered to be the most important piece of bridge work undertaken in Mexico. He remained with this Company until July, 1885, serving for about a year as Bridge Engineer on the southern half of the road and, for another year, as Resident Engineer on the northern 600 miles of the same road, designing and constructing a great many of the bridges.

In September, 1885, Mr. Richardson was appointed Division Engineer on the construction of the Chicago, Burlington and Northern

* Memoir prepared by the Secretary from information on file at the Society Headquarters.

Railroad in Wisconsin, in charge of timber trestle work over the Wisconsin River and also of the heavy grading work. In July, 1886, he entered the employ of the Colorado Fuel Company, making railroad surveys, designing structures, and making estimates for the development of that Company's coal properties in the mountains between Glenwood Springs and Aspen, Colo. He also located a number of railroad lines in this region with grades as high as 5%, using switchbacks on many of the lines. He remained with the Colorado Fuel Company until May, 1887, when he was made Division Engineer on the construction of the Atchison, Topeka and Santa Fé Railroad. In this capacity he had charge of the Division Yard at Chillicothe and the heavy work near the Illinois River. For the greater part of the last year (1888) he served as Resident Engineer in charge of all new work in the State of Illinois.

From January to April, 1889, Mr. Richardson was engaged on the location of a line in the San Luis Valley, from Valley Grove to Alamaso, for the Denver and Rio Grande Railroad, and, from April, 1889, to January, 1892, he was Chief Engineer on the location and construction of the Manitou and Pike's Peak Railroad. This road has a maximum grade of 25 ft. per 100, with a maximum curvature of 16°, is 9 miles long, and extends from an elevation of about 6 000 ft. to one of 14 100 ft. above sea level, about 3½ miles of it being above the timber line. Mr. Richardson designed all the structures and all the appliances for laying the Abt rack rail which was used on this line.

In February, 1892, he went to Chicago, Ill., where he entered the service of the Sanitary District as Assistant Engineer, having charge of the construction of several miles of the Lemont portion of the main channel, and from April to August, 1893, he served as Engineer in charge of the construction of the 68th Street Tunnel (one of the water tunnels built of brick through clay and silt 4 miles under Lake Michigan) for the City of Chicago.

In August, 1893, Mr. Richardson returned to Massachusetts where he entered the service of the State Board of Health and, until June, 1895, had charge of surveys for all the reservoirs and aqueducts in connection with the investigations for a water supply for the Metropolitan District. He made the preliminary locations, designs, and estimates of cost, for the location of the Wachusett Aqueduct; the Wachusett Reservoir and Dam, including all appurtenances thereto; a reservoir on Ware River, together with a tunnel 8.7 miles long connecting it with the Wachusett Reservoir; a reservoir on Swift River, including a tunnel 27.6 miles long, also connecting this reservoir with the Wachusett Reservoir; and the Weston Aqueduct, as well as many other sources of supply.

From July, 1895, to January, 1906, Mr. Richardson was employed as Engineer of the Dam and Aqueduct Department of the Metropolitan

Water-Works, in charge of the construction of the Wachusett Reservoir, 12 miles long, which work included a tunnel 2 miles long and a masonry arch bridge over the Valley of the Assabet River. He also had charge of the construction of the sewage filtering system for Clinton, Mass., including a power-house and 25 acres of filter beds. Until May, 1906, he had charge of studies for the location and design of the North Dike and the land surveys for the Wachusett Reservoir. This work included the relocation of the Central Massachusetts Railroad from Berlin Junction to the North Dike, with a steel viaduct about 135 ft. high and 1 000 ft. long, across the Nashua River below the dam. Mr. Richardson also made the final location for the Weston Aqueduct and was in charge of the construction work in connection with the Wachusett Dam, as well as serving as Engineer of the Reservoir Department in 1905.

In January, 1906, Mr. Richardson entered the employ of the J. G. White Company, Incorporated, as Construction Superintendent in charge at the Laguna Dam near Yuma, Ariz., for the United States Reclamation Service.

In March of the same year, he returned to the Metropolitan Water-Works of Boston, Mass., as Engineer of the Dam and Reservoir Department, which position he held until July, when he was appointed Resident Manager in charge of the construction of the plant of the Rockingham Power Company on the Peedee River at Hamlet, N. C., where he remained until July, 1907.

In September, 1907, Mr. Richardson again joined the forces of the J. G. White Company, Incorporated, and was placed in charge of the La Crosse water-power development at Hatfield, Wis. In December, 1907, he returned to New York City, and on January 1st, 1908, was appointed General Superintendent of Construction in charge of contract work for the New York office. In this capacity, he had charge of various large hydro-electric works, such as the La Crosse Power Company's development, at Hatfield, Wis.; the Connecticut River Power Company's development near Brattleboro, Vt.; the Central Georgia Power Company's development at Jackson, Ga.; the Idaho-Oregon Light and Power Company's development at Copperfield, Ore.; the Minnesota-Ontario Power Company's development at International Falls, Minn., as well as the Idaho Irrigation development in the vicinity of Richfield, Idaho. Mr. Richardson continued as General Superintendent of Construction until September 1st, 1909, when he was made Chief Civil Engineer of the Company. While in this position, he made numerous investigations, studies, and cost estimates for power developments and also acted as Consulting Expert in engineering matters for the Company. During this time he was also

engaged as Consulting Engineer in connection with the Fellsmere Farms Company's work of reclaiming swamp lands in Florida.

When the Engineering Department of the J. G. White Company, Incorporated, was taken over by its subsidiary, The J. G. White Engineering Corporation, January 1st, 1913, Mr. Richardson became Civil Engineer for the latter, which position he retained until August 1st, 1914, when he was obliged to resign on account of failing health. He continued, however, to engage in consulting work until a short time before his death which occurred at his home at Rutherford, N. J., on December 26th, 1915.

Mr. Richardson was engaged for more than 40 years in all kinds of engineering work, including the construction of railroads, water supply, irrigation, drainage, and hydro-electric projects. His work extended over all parts of the United States, which fact made him widely known as a capable engineer, an energetic executive, and a faithful and honest man.

On October 4th, 1888, Mr. Richardson was married to Miss Nellie Symonds who, with three sons, survives him.

He was a member of the Boston Society of Civil Engineers, the New England Water Works Association, and the Engineers' Club of New York City. He was also a Knight Templar and a Mystic Shriner.

Mr. Richardson was elected a Member of the American Society of Civil Engineers on November 4th, 1885.

ROBERT MAITLAND ROY, M. Am. Soc. C. E.*

DIED JUNE 27TH, 1916.

Robert Maitland Roy, the son of Robert Maitland and Maria Roy, was born at Stirling, Ont., Canada, on November 27th, 1869. He was educated in the public and high schools of Belleville, Ont., and was graduated from Ontario Commercial College. He also spent a year or more in the study of commercial and contract law, and took a special private course in mathematics.

In June, 1886, Mr. Roy entered the employ of the Grand Trunk Railway, serving, until October, 1888, in the Traffic and Engineering Departments.

From October, 1888, to February, 1892, he was engaged as Assistant Mechanical Engineer with the G. and J. Brown Manufacturing Company, of Belleville, Ont., inspecting Government and railway supplies, contractors' plant for public works, railway, mining, and other contracts, as well as structural work of various kinds. Mr. Roy was also engaged during this time on Government surveys of the Belleville Harbor and River protection works, on extensions to the Belleville Water-Works, and on new sewerage and pumping plants for the Government Institution for the Deaf and Dumb at Belleville.

From February, 1892, to January, 1898, Mr. Roy was employed with the Central Bridge and Engineering Company, of Peterborough, Ont., as Assistant and Contracting Engineer for railway and highway bridges and general structural work. He was also engaged during this time on surveys of the Trent Canal and other works.

In January, 1898, Mr. Roy entered the employ of the Hamilton Bridge Works Company, Limited, of Hamilton, Ont., serving as Highway Bridge Engineer from 1898 to 1908, as Manager, from 1908 to 1916, and as Director from 1913 to 1916, which latter positions he held at the time of his death, on June 27th, 1916.

Mr. Roy was a Member of the Canadian Society of Civil Engineers, the Hamilton Board of Trade, and the Canadian Manufacturers Association, and had served as an Alderman of Hamilton, Ont., from 1913 to 1916. He was also a member of the Hamilton, Twentieth Century, Canadian, and Hamilton Automobile Clubs, the I. O. O. F., the A. F. and A. M., and of the Church of England.

On April 19th, 1894, Mr. Roy was married to Mary Ann Wragg, daughter of Thomas B. Wragg, of Belleville, Ont., who, with four sons, survives him.

Mr. Roy was elected an Associate Member of the American Society of Civil Engineers on October 3d, 1900, and a Member on March 5th, 1907.

* Memoir prepared by the Secretary from information supplied by J. A. Spittle, Esq.

LEONARD WARREN RUNDLETT, M. Am. Soc. C. E.*

DIED OCTOBER 13TH, 1916.

Leonard Warren Rundlett was born in Alna, Lincoln County, Me., on September 21st, 1846. During boyhood he lived in Brunswick, Me. While he was very young his father died, after which he lived with his widowed mother and only sister. He attended school at Brunswick, and was graduated from Bowdoin College in 1868, afterward taking a post-graduate course in engineering.

Two years after graduation he went to St. Paul, Minn., where he joined the Engineer Corps of the St. Paul and Pacific Railroad Company (now the Great Northern Railway), and was engaged for two years in railroad construction work in Minnesota.

In 1872 Mr. Rundlett entered the employ of the City Engineer of St. Paul. He remained in this office, succeeding to that position nine years later. Resigning from that office in 1883, he devoted two years to planning and constructing the St. Paul Water-Works. He then resumed the position of City Engineer and remained in charge of the engineering work of the city, with the exception of one term, until 1911. During this time the city had increased from a frontier town of 25 000 people to a city of 250 000. Its public works, bridges, streets, sewers, water-works, and public buildings, costing many millions, had been planned and executed under his supervision. The administration of the Engineering Department had occupied practically all his professional life, as he had been connected with it for 38 years; during this time the city government was controlled by parties of different political views, but, in the words of the leading paper of the city after his death:

"Through them all L. W. Rundlett was recognized as a competent engineer and a personally honest man. He came through all without a taint upon his personal honesty."

The Commercial Club of the city, of which he was an active member for many years, in memorial resolutions states:

"During many years of work as a public official of this city he exemplified the highest type of official service, marked by ability, integrity, and humanity; and was in all things above the temptations of office, power, or place; caring not for public or personal honors so much as he cared for the approval of his own conscience and the merited affection of his fellow-men."

During the term he was not City Engineer, Mr. Rundlett was in charge of important hydraulic development on the Apple River, Wisconsin.

* Memoir prepared by George L. Wilson, M. Am. Soc. C. E.

In 1913 and 1914 he was Commissioner of Public Works for the City of Moose Jaw, Saskatchewan, Canada. After this, he opened an office as Consulting Engineer in St. Paul. His health had been poor for some months; he had been suffering from heart trouble, but finally, on October 13th, the end came suddenly at his home.

He was married to Miss Mary Atwater Barry, of Milwaukee, Wis., on October 27th, 1881. Miss Barry was the daughter of Capt. Garrett Barry, a graduate of West Point, and Mary Atwater, of Wallingford, Conn. Mrs. Rundlett and one daughter, Mrs. Garritta (Rundlett) Edgerton, survive him.

Mr. Rundlett was elected a Member of the American Society of Civil Engineers on September 5th, 1883. He served as a Director of the Society in 1911-13.

FRANK EDSON SHEDD, M. Am. Soc. C. E.*

DIED SEPTEMBER 22D, 1916.

Frank Edson Shedd was born in Sharon, N. H., on July 18th, 1856. He attended the Conant High School of East Jaffrey, N. H., and entered Dartmouth College, from which he was graduated in the Class of 1880.

After a year of teaching, as Principal of a high school, Mr. Shedd entered the service of the United States Coast and Geodetic Survey, where he remained for a year, his work being on the charting of the coast of Maine. In 1882, he left the service of the Government to take up civil engineering in Lowell, Mass.

In 1886, Mr. Shedd had charge of the erection of the Washington Mills, at Lawrence, Mass., at that time one of the largest mill construction propositions which had ever been undertaken. These mills are now owned by the American Woolen Company.

In April, 1887, he became a member of the staff of Lockwood, Greene and Company, the engineering firm which had designed the Washington Mills, and two years later was made First Assistant to Mr. Stephen Greene, then the sole member of the firm. From that time until his death, Mr. Shedd maintained an intimate connection with that organization.

On January 1st, 1901, on the incorporation of Lockwood, Greene and Company, Mr. Shedd became a Director and the Vice-President of the firm, and held both these positions until his death. He was a Civil Engineer of high standing, and had designed many large mills and hydraulic plants in various parts of the United States and Canada, having been considered one of the leading authorities in this country on hydraulic developments.

Mr. Shedd was a man of varied interests; a member of the Boston Society of Civil Engineers and of the American Society of Mechanical Engineers, he was also a member of St. John's Lodge, A. F. and A. M., of Boston, a Companion in the Mount Vernon Royal Arch Chapter, and a member of the Second Parish Congregational Church, of Dorchester, Mass.

Mr. Shedd was also a member of the New Hampshire Historical Society and a life member of the New England Historic Genealogical Society. He was the founder, as well as the Secretary and Treasurer, of the Shedd Family Association, and designed the handsome monument—erected at Shedd's Neck, in Quincy, Mass., three weeks prior to his death—by the Shedd Association, in honor of the first ancestor of

* Memoir prepared by Mr. Frank W. Reynolds, of Lockwood, Greene and Company, Boston, Mass.

the family in America. Having been for many years much interested in genealogical research, he had collected many valuable data regarding the Shedd descendants in America, and was making plans, when taken sick, looking toward the early publication of a genealogy of the family.

Mr. Shedd was a thorough, painstaking engineer, of retentive memory, wise judgment, and tireless industry; a modest unobtrusive gentleman of sterling character. One of his friends says of him:

"He was a high-minded citizen and represented the highest ideals of human life; he rendered a large service to his fellow-men; he was an honor to his Profession and has left behind him a name which ought to be an inspiration and help to us all for higher and better things."

During his illness, which covered the last six months of his life, Mr. Shedd suffered much, but bore his sufferings with great fortitude, and attended to many affairs up to the very last. He died at his home in Dorchester, Mass., on September 22d, 1916, leaving a widow and one son.

Mr. Shedd was elected a Member of the American Society of Civil Engineers on February 6th, 1907.

DAVID SIMSON, M. Am. Soc. C. E.*

Died December 16th, 1916.

David Simson was born at Bedrule House, Roxburghshire, Scotland, on November 9th, 1861. He was educated at St. Andrew's and Edinburgh University, and from September, 1877, to June, 1879, served as Apprentice Mechanical Engineer in the shops of Messrs. Douglas and Grant, at Kirkcaldy, Scotland, where he was engaged on the construction of large stationary engines, boilers, sugar and paper machinery, etc. From February, 1880, to January, 1882, he also worked as an Apprentice to Messrs. Carfrae and Belfrage, Land Surveyors, of Edinburgh, Scotland, and was engaged in surveying, the construction of roads, sewers, etc.

Having served his apprenticeship, Mr. Simson was employed, from January, 1882, to July, 1884, by Messrs. J. Waddell and Son, Railway Contractors, of Edinburgh, as Contractor's Engineer in charge of the construction of the eastern half of the Edinburgh Suburban Railway. This work involved the deviation of streets, sewers, etc., and the crossing of the new railway over and under existing railways and canals.

From August to November, 1884, Mr. Simson traveled over the greater part of the United States and Canada, in order to study American methods of railroad construction. On his return to his home, he was engaged, from January to June, 1885, by the creditors of Messrs. L. and K. Macdonald, Contractors, to finish the construction of the Killin Railway, in Perthshire, Scotland. One of the most important parts of this work was the construction of a viaduct over the River Dochart. This viaduct consisted of five skew spans of about 50 ft. each, the arches being of concrete. This is the first instance in which concrete was used for arches of this magnitude in railway construction in Great Britain.

In June, 1885, Mr. Simson entered the employ of John Strain, Civil Engineer, and was engaged in making surveys and preparing drawings for the various railway construction projects on which the firm was working. He also assisted Mr. Strain in giving evidence before Parliamentary Commissions in London, and in 1887-88, served as Resident Engineer on the construction of the Kilbirnie Branch of the Lanarkshire and Ayrshire Railway, for which he had previously made the surveys and drawings.

In September, 1888, Mr. Simson resigned his position with Mr. Strain, and went to South America where he was appointed Chief Assistant Resident Engineer on the Mountain Division of the Buenos Aires and Valparaiso Transandine Railway from Mendoza. This Division covered 100 miles of extremely difficult mountain railway

* Memoir prepared by the Secretary from information on file at the Society Headquarters.

construction, especially in connection with the gorge of the Mendoza River in which the roadbed had to be blasted from the sides of the precipice. Mr. Simson remained in this position until November, 1891, at which time the line was constructed and open as far as Rio Blanco, and work on the Summit Tunnel had been commenced.

He then returned to Scotland, but, in October, 1892, he was appointed Chief Engineer of the Western Railway of Havana, in Cuba, which position he held until June, 1896. During his incumbency, all the old timber trestles along the line of this road were replaced by steel structures, and the railroad was extended 10 miles to Pinar del Rio, into the heart of the best tobacco district on the island.

In November, 1896, Mr. Simson entered the service of the Buenos Aires Western Railway Company as Chief Engineer, with headquarters at Buenos Aires. In September, 1897, he was appointed Acting Manager, and in September, 1898, General Manager of the Company, which position he held until December, 1906. On his retirement, the employes of the Railway Company presented him with a silver model of a locomotive in token of their affection and respect.

From January to March, 1907, Mr. Simson was engaged in Chile and Bolivia, in making a report on the affairs of the Antofagasta and Bolivia Railway. In April of that year, he retired from the active practice of engineering and returned to England to make his home.

Subsequently, he was elected a member of the Board of Directors of the Buenos Aires Western, the Antofagasta and Bolivia, and the Great Western of Brazil Railways, but he was obliged to retire from the latter two, in 1912 and 1913, respectively, on account of pressure of other work. He had also been elected, in November, 1907, a Director of the Buenos Aires Great Southern Railway Company, and was made Chairman of the Board in September, 1910. He was also a Director of the Buenos Aires Southern Dock Company and of the London and River Plate Bank.

At the time of his death, which occurred suddenly by heart failure at his home at Ickleford Manor, Hitchin, Herts, Mr. Simson was actively engaged in war relief work in his district.

He was a member of the Institution of Civil Engineers of Great Britain, and a "Caballero" of the Second Class of the Spanish Order of Military Merit, having been decorated for services rendered the Spanish Government in conducting armored trains and repairing the Western Railway of Havana under fire during the Cuban Rebellion of 1896.

He was widely versed in South American affairs, and his death will be deeply felt by the Anglo-Argentine Colony of which he was one of the best known and most valued members. He is survived by his widow and one son.

Mr. Simson was elected a Member of the American Society of Civil Engineers on January 8th, 1902.

FRANK OSCAR SINCLAIR, M. Am. Soc. C. E.*

DIED NOVEMBER 15TH, 1916.

Frank Oscar Sinclair was born at Burlington, Vt., on September 7th, 1860. He received his technical education at the University of Vermont, from which he was graduated in the Class of 1882.

During the summers of 1881 and 1882 he was employed as Assistant Engineer on the location and construction of the Canada-Atlantic Railway. In May, 1883, Mr. Sinclair went West and was engaged until July, 1884, as Assistant City Engineer of Leavenworth, Kans., in charge, under the late G. W. Pearsons, M. Am. Soc. C. E., as Consulting Engineer, of the construction of a sewer system for that city.

From July, 1884, to October, 1892, he was engaged on railroad work in the West and South, as follows: July, 1884, to January, 1885, Assistant Engineer with the Missouri Pacific Railway Company, with headquarters at Atchison, Kans.; July, 1885, to July, 1887, Assistant and Division Engineer with the Chicago, Rock Island, and Pacific Railway Company, in charge of part of location and construction on lines west of the Missouri River; September, 1887, to July, 1890, Principal Assistant Engineer of the Knoxville Southern Railway; July to November, 1890, on location for the West Virginia and Pittsburgh Railroad Company in West Virginia; August, 1891, to October, 1892, Chief Engineer for the Ducktown (Tenn.) Mineral Railway Company and the Ducktown Sulphur, Copper, and Iron Company, constructing a narrow-gauge railway, erecting a crushing plant, smelting sheds, roasting furnaces, etc.

Mr. Sinclair then returned to Burlington, Vt., where he opened an office and engaged in private practice as a Consulting Engineer. He served as Chief Engineer on the construction of the Burlington Traction Company's lines, the water-power plant for Vergennes, and the Bolton Falls Dam. He was also engaged, from June, 1896, to August, 1900, as Contracting Engineer for the Pittsburgh Bridge Company in Vermont.

In 1904, Mr. Sinclair was appointed City Engineer of Burlington, Vt., and held that office until his death. He also served as a member of the Board of Aldermen from Ward 1, from 1899 to 1903; Superintendent of the Water Department from 1905 to 1909; and Member of the Board of Street Commissioners from 1913 to 1916, serving as Chairman in 1913 and 1914.

In 1910, Mr. Sinclair was appointed Consulting Engineer of the Public Service Commission of Vermont, which position he held at

* Memoir prepared by the Secretary from information on file at the Society Headquarters.

the time of his death which occurred on November 15th, 1916, following an operation. He is survived by his widow and three children, and by two brothers.

Mr. Sinclair was a man of the conservative type. He had many times proved himself capable of grasping large problems and of working them out successfully. He had established a reputation as a careful and accurate engineer, in recognition of which he had filled many positions of trust, among which was a recent appointment by the Secretary of the Navy as Chairman of the State Directors and Associate Members of the Naval Construction Board of Vermont. His sudden death was a distinct shock to the people of Burlington, which city he had served worthily in so many capacities, and his loss will be deeply felt by all who knew him.

He was a Past-President of the Vermont Society of Engineers, a member of Hamilton Lodge, I. O. O. F., the Algonquin Club, and a charter member of Vermont Alpha of Phi Delta Theta. He was also a member of the Official Board of the Methodist Church, and had been a teacher in the Sunday School for many years.

Mr. Sinclair was elected a Member of the American Society of Civil Engineers on November 6th, 1901.

DAVID WENDEL SPENCE, M. Am. Soc. C. E.*

DIED JUNE 28TH, 1917.

On September 22d, 1868, David Wendel Spence, the son of the late Joseph and Margaretta (Wendel) Spence, was born at Austin, Tex. His early education was obtained at private schools in Austin, and, for four years, he was a member of the classes in the late Professor Bickler's Academy. In June, 1889, he received his B. Sc. degree from the then young University of Texas, his work as a student having been in the Civil Engineering course.

During the summer of 1889, Mr. Spence was a member of the geological field party of the late Dr. Theodore B. Comstock, then Geologist in charge of the investigation of the Central Mineral District of Texas, on the Texas State Geological Survey. In September, 1889, he entered the University of Michigan where he pursued advanced work in civil engineering under the late Charles E. Greene, M. Am. Soc. C. E., then Professor of Civil Engineering in that institution, and spent one year in resident study. In 1891, after having had one year's active practice in his profession, at Denver, Colo., he received his C. E. degree from the University of Michigan.

Mr. Spence's year in Denver was spent in the office of the Lane Bridge and Iron Works, the factory of which company was in Chicago, Ill. Later, he returned to Chicago, and was Chief Draftsman of the South Halsted Street Iron Works until September, 1892, when he resigned to return to Texas to accept a position in the Agricultural and Mechanical College of Texas, with which institution he maintained continuous connection until his death. During his twenty-five years of service at the College, he occupied, successively, the following positions: 1892 to 1898, Assistant Professor of Civil Engineering and Physics, and of Drawing; 1898 to 1899, Associate Professor of Civil Engineering and Physics, and Drawing; 1899 to 1904, Professor of Physics; 1904 to 1908, Associate Professor of Civil Engineering; 1908 to 1913, Professor of Structural Engineering; and, 1913 to 1917, Dean of the School of Engineering, and Professor of Civil Engineering.

Dean Spence organized the Texas Engineering Experiment Station, at the Agricultural and Mechanical College, and was its Director from its inception until his death. He also rendered additional valuable service to the College along several lines of physical development. From 1907 to 1911, he was in charge of repairs to buildings, and of the maintenance and extension of the water supply and sewer systems of that institution. He was also Supervisor of Construction of the $100 000 mess hall, built at the College in 1913. From 1912 until

* Memoir prepared by J. C. Nagle, M. Am. Soc. C. E.

the time of his death, Dean Spence was Supervising Engineer for all new construction work at the Prairie View, Tex., Normal and Industrial College (for negroes), over which the Board of Directors of the Agricultural and Mechanical College has jurisdiction. From 1911 until his death, he had complete charge of the repairs to buildings, and of water-works and sewer extensions, at Prairie View.

During his vacations, Dean Spence occupied various positions, among which the following may be mentioned: 1894, Assistant Engineer, Brazos and Burleson Railway Company; 1895 and 1896, Engineer, Brazos River Channel and Dock Company, Velasco, Tex.; 1899, Draftsman, Union Bridge Company, Athens, Pa.; 1900, Assistant Engineer, Houston, East and West Texas Railroad Company; 1901 Draftsman, Columbia Bridge Company, Carnegie, Pa.; 1903, Assistant Locating Engineer, Gulf, West Texas and Pacific Railroad Company; and 1905, Draftsman, Penn Bridge Company, Beaver Falls, Pa.

Dean Spence also did considerable private work as Consulting Engineer, especially on highway bridges.

Among his former students are numbered many prominent engineers, not only of Texas, but of other States and of foreign countries, every one of whom bears him in affectionate remembrance, and who, together with a host of other friends, deeply mourn his untimely end. Zeal for his work, and love for his institution, caused him for years to overtax himself, and to disregard the warnings of failing health. Almost up to the very hour of his death, he was busy planning betterments in his college courses and improvements in the conduct of his Department. Long will the College miss his wise counsel and energetic efforts in its behalf.

In June, 1893, Mr. Spence was married to Miss Lucy Reese, daughter of the late distinguished Judge T. S. Reese, at Hempstead, Tex., who, with one son (who also is a civil engineer), and two daughters, survives him.

Dean Spence was elected a Member of the American Society of Civil Engineers on October 1st, 1913.

HENRY GORDON STOTT, M. Am. Soc. C. E.*

DIED JANUARY 15TH, 1917.

Henry Gordon Stott was a native of the Orkney Islands, Scotland, where he was born on May 13th, 1866, the son of the Rev. David and Elizabeth Jane (Dibblee) Scott. After a thorough grounding in the fundamentals, by his father and elementary school instructors, he was enrolled as a student at the Watson Collegiate School, Edinburgh. On leaving this institution he entered the College of Arts and Sciences at Glasgow, and began a course in mechanical engineering and electricity, being graduated in 1885. In the year previous he had entered the employ of the Electric Illuminating Company, of Glasgow. Shortly after his graduation he was made Assistant Electrician on board the Steamship *Minia,* belonging to the Anglo-American Telegraph Company. The next 4½ years saw him engaged with those duties, during the course of which he saw much service in connection with repairs to the cable lines of that company. During this period he undertook a number of experiments that resulted in the introduction of improved methods of handling cable repairs. He was also identified with the "duplexing" of the United States Cable Company's main cable (2 750 knots), the longest duplex cable in the world.

In 1889 Mr. Stott was made Assistant Engineer of the Brush Electric Engineering Company's plant at Bournemouth, England. In the following year he accepted a position with Hammond and Company as Assistant Engineer in the construction of an underground cable and power plant at Madrid, Spain. He remained there until 1891, coming to the United States in that year to construct an underground cable and conduit system for the Buffalo Light and Power Company (now the Buffalo General Electric Company). This work was completed with a degree of success that reflected very great credit on Mr. Stott. As a result he was named Engineer of the company, and during the next 10 years was one of the most active figures in the industrial progress of Buffalo. During this period he designed and executed some notable construction work, including a power-plant on Wilkeson Street, Buffalo.

His efforts attracted wide attention, and in 1901 he was appointed Superintendent of Motive Power of the Interborough Rapid Transit Company, New York City, a position which he filled with signal success. At the time he took up these duties the Interborough had not yet been organized, the company having the title of the Manhattan Railway Company. The post to which Mr. Stott was called had just

*Memoir prepared by W. S. Finlay, Jr., Esq.

been created, and it devolved upon him to organize the operating force, in connection with which he completed the 74th Street power-plant of the company, and various sub-stations and transmission lines.

When the Manhattan System was amalgamated with the Interborough, in 1904, Mr. Stott was invited to retain his office with the new corporation. He accepted, and immediately took over supervision of the construction of the power-plant on 59th Street. Since that time he had been constantly in charge of the design, construction, and operation of the power generating stations and the distributing system of the Interborough, which comprehends both the subway, elevated, and surface lines of New York City.

The plans for the electric power system of the new subway lines were developed under his supervision, and the work has progressed so far and bears so strongly the stamp of his work that, when completed, it will be a monument to him.

Mr. Stott was a firm believer in co-operation among engineers, through the agency of engineering societies. He was elected President of the American Institute of Electrical Engineers for the term 1907-1908, Vice-President of the American Society of Mechanical Engineers for the term 1912-1914, Director of the American Society of Civil Engineers in 1911, and was Vice-President and Trustee of the United Engineering Society at the time of his death. Up to the last, Mr. Stott was a recognized power in the American Institute of Electrical Engineers, and was a member of the Standards Committee, the Committee on Development of Water-Power, the United States National Committee of the International Electrotechnical Commission, the Power Stations Committee, the Public Policy Committee, the Edison Medal Committee, the Committee on Economics of Electric Service, and was one of the Institute's representatives on the Joint Committee on the Metric System, of which he was an ardent advocate.

As a result of his unusually wide experience and extended research, Mr. Stott was called on often to contribute papers to the various engineering societies. He was especially well known for his minute analysis of enginering problems. Among the many papers written by him on this subject are "The Conversion and Distribution of Received Currents", "Power Plant Economics", "Notes on the Cost of Power", "Steam Pipe Covering and Its Relation to Station Economy", "Tests of a 15 000-Kilowatt Steam Engine Turbine Unit", "Power Plant Design and Operation" (a series), etc.

Mr. Stott was a remarkable figure in the engineering world, because he was in the front rank of both electrical and mechanical engineers, because, in both branches of the art, he was a master of theory and practice, and because, with these technical qualifications, he combined a rare executive ability, a power of inspiring the confidence of his

employees, and of bringing out the best that was in the men who worked for him.

Mr. Stott's activities were not confined to engineering matters. He early became a citizen of the United States and served for 5 years in the 74th Regiment of the National Guard of New York State.

He was also an active member of the Protestant Episcopal Church, being a communicant of St. Paul's Church, of New Rochelle, in the affairs of which he was deeply interested.

Mr. Stott was married on July 22d, 1894, to Miss Anna Mitchell, of Belfast, Ireland, who, with their two children, a son and a daughter, survives him. He died at his home, in New Rochelle, N. Y., on January 15th, 1917, after an illness of many months.

Mr. Stott was elected a Member of the American Society of Civil Engineers on July 1st, 1908.

SLEDGE TATUM, M. Am. Soc. C. E.*

DIED JANUARY 18TH, 1916.

Sledge Tatum, the son of Seth and Sarah Elizabeth Tatum, was born on July 10th, 1870, at LaGrange, Ga. He received his education at the LaGrange High School and from private instructors.

He began his professional career in 1886 in the Engineering Department of the Macon and Birmingham Railroad, and, later, was engaged in surveying and engineering work in Fulton and Troup Counties, Georgia.

In 1894 he was elected a member of the Georgia Legislature to represent Troup County.

In May, 1895, Mr. Tatum was appointed Surveyor in the United States Geological Survey, and spent several years surveying land lines in Indian Territory. In 1899, he was appointed Topographer, and carried on triangulation in various States. In 1903, he surveyed forest boundaries in Washington and Idaho, and the following year executed triangulation along the International Boundary.

On June 10th, 1905, he was transferred from the Department of the Interior for service with the Department of Engineering of the Isthmian Canal Commission. He began work in the Canal Zone as Instrumentman on the Chagres River surveys, and advanced rapidly through the different grades, at one time having charge of all the survey parties in the field, both on the Chagres River Division and the Canal Zone boundary. On November 1st, 1908, Mr. Tatum was made Superintendent of Construction and was employed on the building of the Gatun Dam under Gen. William L. Sibert, M. Am. Soc. C. E., Division Engineer. He had charge of the surveys, the excavation at the spillway, and the driving and filling of all trestles. He resigned on February 8th, 1909, to accept a transfer to the United States Geological Survey.

On his return to the Survey, Mr. Tatum was assigned to special investigations and, on June 1st, 1910, was appointed Geographer in charge of the Rocky Mountain Division, comprising North and South Dakota, Kansas, Nebraska, Oklahoma, Montana, Wyoming, Colorado, New Mexico, and Texas, with summer headquarters at Denver, Colo. His duties comprised the planning and supervision of all topographic mapping in this area.

On December 10th, 1914, he was appointed Acting Chief Geographer of the Topographic Branch of the United States Geological Survey, in which capacity he served until his death.

* Memoir prepared by Frank Sutton, M. Am. Soc. C. E.

Mr. Tatum was one of those rare combinations of thorough ability, good judgment, and consideration of the rights of others, and his death was a great loss, both to his associates in the Survey, with whom he had served for twenty years, and to his wide circle of friends.

C. L. Carpenter, M. Am. Soc. C. E., Manager of the Central Aguirre Company, of Central Aguirre, Porto Rico, writes of Mr. Tatum as follows:

"I believe there was hardly a person on the Isthmus during the time he was there who was so universally respected and liked by the people to whom he reported and by the people who reported to him, and I can hardly think of a person on the Isthmus to whom this would apply so well during the four years that I was connected with the Panama Canal. He certainly was a very loyal employee of the Isthmian Canal Commission and a very loyal friend. He was an extremely good judge of human nature and a good executive, and he did a great deal to help in training many young men while he was on the Isthmus.

"I know very few men who had all Mr. Tatum's good qualities, and it is certainly a pity that he should be called away so soon."

Mr. Tatum was a member of the Washington Society of Engineers and of the Cosmos Club of Washington. He also belonged to a Mississippi Commandery of Knights Templar.

He was married on May 5th, 1909, to Miss Sarah Richardson, of Huntsville, Ala., a daughter of Representative William Richardson and Elizabeth Benagh Rucker Richardson, and is survived by his wife.

Mr. Tatum was elected a Member of the American Society of Civil Engineers on January 4th, 1910.

EDWARD CLINTON TERRY, M. Am. Soc. C. E.*

DIED APRIL 6TH, 1908.

Eli Terry, of Plymouth, Conn., began to make mantel clocks in 1814, and these soon drove other styles out of the market. A few of these mantel or shelf clocks, with wooden wheels and bearing the legend "Warranted if well used", are still sometimes found in Connecticut. His son Eli continued the manufacture of clocks at Terryville, Conn., a village which he founded about 1825, and his grandson James inherited the business, later turning it into the manufacture of locks for trunks. James Terry was of a highly mechanical turn of mind; he installed a pipe organ and a water motor in his house, and also made a working model of a steam road vehicle which walked on four legs like a horse.

Edward Clinton Terry, the youngest child of James and Elizabeth (Hollister) Terry, was born at Terryville on December 10th, 1850. As a lad, he had the advantage of learning much from his father regarding machinery and hydraulic motors. He prepared for college at the High School, in Hartford, Conn., and took the Civil Engineering course at the Sheffield Scientific School of Yale University, from which he was graduated in 1871. As a student, his work was well and faithfully done, and among his classmates he was regarded as a clear thinker, fond of philosophical discussions, and as an excellent chess player.

During 1871-73, Mr. Terry worked as a Rodman on the Easthampton Branch of the Connecticut River Railroad, and was also engaged on surveys of the reservoirs for the water supply of Hartford, Conn. He then spent much time in the study of water meters, and took out patents for both a rotary and a piston meter. The first of these had a wabbling disk which caused the rotation of an axis that acted on the recording apparatus; this he called the "Undine Meter", the name being taken from a German story which was read by his class in the Freshman year. In 1880, he became connected with the Hartford Meter Company, with which he was associated until 1889, at first as General Manager and Secretary, and, later, as its proprietor.

In 1890, Mr. Terry was one of the organizers of the Farmington River Power Company, and, as its Engineer, he designed and constructed a plant for generating electric power on the Farmington River and transmitting it a distance of 11 miles to Hartford. This work included a dam 500 ft. long and 18 ft. high, the hydro-electric power plant, and the transmission line. It delivered at first 800 h.p. to the

* Memoir prepared by Mansfield Merriman, M. Am. Soc. C. E.

Hartford Electric Light Company, and, later, this was increased to 1 300 h.p. This plant is said to have been the first long-distance transmission of power by electricity in America. He was Secretary of the Company and Manager of its engineering work for several years, and it was during this period that modern methods were first used in generating electric energy from water power and in transmitting it to Hartford.

About 1888, Mr. Terry became interested in steam turbines and, in the following ten years, took out patents for three of high speed and three of low speed. In 1906, he organized a company for the manufacture of low-speed turbines, of which he was President until his death. Two of the first turbines were installed in New York City at the Waterside Stations of the Edison Company, and were used to drive boiler-feed pumps; in 1917 there are 24 such steam turbine-driven pumps in use at these Stations. Altogether, about 3 300 Terry turbines have been made and put into operation since 1906.

Mr. Terry had a wide reputation as a man of mechanical genius and high business capacity. Although rather retiring in manner and of a modest disposition, he was cheerful in temperament. Those who became his friends always remained such, admiring his independence of thought, his power of keen analysis of political and philosophical problems, and his sterling character.

In college, Mr. Terry was a member of the Book and Snake Fraternity and of the Sheffield Chess Club. In Connecticut, he was a member of the Hartford Club and of the Golf Club. In politics, he was always an independent voter. In 1901, he went to Europe, but his inclination for travel was never great, and he preferred the quiet of his home and the conversation of his family and friends. He died on April 6th, 1908, at his residence in Hartford, Conn., following a week's illness of pneumonia.

Luther W. Burt, Esq., formerly City Engineer of Hartford, writes as follows:

"Terry always met an acquaintance with a cordial greeting and a pleasant smile. His conversation was always informing and never intrusive. I cannot remember that he ever spoke a disparaging word of any one. He was upright, studious, and self-centered. He had, I think, few intimates, but he generously recognized the services of friends and employees."

Henry W. Sargent, Esq., Vice-President of Sargent and Company, of New Haven, Conn., writes:

"Graduating as a Civil Engineer, Terry's inherited mechanical instincts led his imaginative and reflective mind into mechanical and electrical fields, which he enjoyed and enriched. With this taste and with capacity for practical invention, he possessed executive qualities,

in a nice command of others, with patience to lead his associates to reason and work with him in the direction of his choice. Such power, strengthened by personal morality and a high ethical sense, makes for a leader of men.

"Terry's nervous energy too early wore out a not too strong physique, and he was called to his fathers at a time when his exertions had just placed his manufacturing company on the firm foundation of developing and supplying what was wanted in a field of prime movers all its own."

James Shepard, Esq., of New Britain, Conn., who was Attorney for Mr. Terry in taking out his patents for water meters and steam turbines, writes as follows:

"My acquaintance with 'Clinton Terry' (as he was generally called), began about 1878, when he became my client. He was one of the most interesting clients I ever had, and it was always a pleasure to meet him. He generally had some useful information to impart, oftentimes upon matters other than the main subject of our interviews. He was cheerful, energetic, diligent, persistent, and generally successful in all his undertakings. He was free to express his opinion as to what he wanted, but he always did so in a courteous and pleasant manner. He was a man of good habits, strictly honest, and prompt in all business matters. His inventions were always fully worked out, even as to details, and reduced to working drawings, before making applications for patents. He was liked by all with whom he came into contact, and his influence upon others was of an elevating character.

"His persistency enabled him to overcome obstacles that would have discouraged other men. As an illustration of this, he came into possession of the first clock model made by his great-grandfather, Eli Terry, after a lengthy controversy and the payment of $1 000 for the clock."

Mr. Terry was married on February 28th, 1872, to Miss Louise Ellen Webster, of Terryville, Conn., who survives him. They had two children: Charles Webster Terry, who died in 1886; and James Terry, who was graduated from the Sheffield Scientific School in 1895, succeeded his father as President of the Terry Steam Turbine Company, and died in 1917.

Edward Clinton Terry was elected a Member of the American Society of Civil Engineers on February 6th, 1895.

ERLE LEROY VEUVE, M. Am. Soc. C. E.*

DIED MARCH 25TH, 1917.

Erle Leroy Veuve was born at the Military Post, at Port Townsend, Wash., on January 21st, 1877. His early education was received at Post Schools in many different parts of the country.

In 1893 he went to work as Chainman for the Northern Pacific Railway Company and attained the position of Levelman. In the fall of 1895, he entered Troop Polytechnic Institute, at Pasadena, Cal., with the intention of taking up the profession of Civil Engineering. In 1896 he entered the Civil Engineering Department of Stanford University, where he continued his studies until 1900. During this time, Mr. Veuve's summer vacations were spent in engineering work on the Northern Pacific and Southern Pacific Railways, and for irrigation companies in the San Joaquin Valley, California. During a part of 1898 he served as Chief Yeoman in the United States Navy.

During the latter part of 1900 and the early part of 1901, Mr. Veuve had charge of a number of short pieces of work. In March, 1901, he became Engineer in charge, for the United Verde and Pacific Railway Company in Arizona, superintending the extensive reconstruction on that road. On the completion of this work, in April, 1902, he was employed as Assistant Chief Engineer of the Pacific Electric Railway Company, in Los Angeles, Cal., first in charge of location and, later, of construction. Mr. Veuve handled much of the extensive development work required in the construction of the Company's inter-urban system. In addition to the railroad lines, this work covered a wide field of engineering and construction.

In 1908, further extensions of the Pacific Electric Railway having been discontinued, Mr. Veuve resigned and went into private practice as Consulting Engineer, with offices in Los Angeles. He was engaged in this practice at the time of his death, which occurred very suddenly and unexpectedly on March 25th, 1917, from cardiac asthma. During the last two years of his life, he had been associated with the writer, under the firm name of Veuve and Strong. As a Consulting Engineer, Mr. Veuve's practice covered a wide field, though, in the main, it was connected with the preliminary investigations of electric railway, hydro-electric, and irrigation projects, much of which was of a confidential character. His ability in this field was well known, and his loss will be felt by an exceptionally large circle of friends and business associates.

* Memoir prepared by A. M. Strong, Assoc. M. Am. Soc. C. E.

On October 27th, 1908, Mr. Veuve was married to Miss Anna M. Heim, of Santa Rosa, Cal. He is survived by his widow, his mother, a son, and a daughter.

Mr. Veuve was elected a Junior of the American Society of Civil Engineers on September 3d, 1901, and a Member on February 2d, 1909. He was also an Associate Member of the American Institute of Electrical Engineers, a member of the Engineers and Architects Association of Southern California, and of the Jonathan Club, of Los Angeles, Cal.

THEODORE VOORHEES, M. Am. Soc. C. E.*

DIED MARCH 11TH, 1916.

Theodore Voorhees, eldest son of Benjamin F. Voorhees and Margaret E. Sinclair, was born in New York City on June 4th, 1847. Mr. Voorhees was a direct descendant in the male line from Stephen Coert Van Voorhees, who came to America from Holland, settling at Flatlands, Brooklyn, N. Y., in 1660. His mother was of Scotch parentage, the Sinclairs having come to America in 1835.

He received his education at Anthon's Grammar School, New York City, Columbia University (where he entered the class of 1868, but left in full standing in 1866), and the Rensselaer Polytechnic Institute, at Troy, N. Y., which he entered in 1866 and from which he was graduated in June, 1869. One month later Mr. Voorhees entered the service of the Delaware, Lackawanna and Western Railroad, as Engineer on the extension from Great Bend to Binghamton. So successful was his work and so progressive were his methods, that in less than 2 years he was made Superintendent of the Syracuse, Binghamton and Oswego Division of this railroad. He held this position for 2 years, and then returned to the Engineering Department of the main road, with headquarters at Scranton, Pa.

Although the love of engineering was undoubtedly responsible for this move, Mr. Voorhees' ability as an operating official, which continued throughout his life, had already developed, and this drew him into the Transportation Department of the Delaware and Hudson Canal Company, in December, 1874. Three months later he was made Superintendent of the Saratoga and Champlain Divisions, a position which he held for more than 10 years. He resigned on October 20th, 1885, to become Assistant General Superintendent of the New York Central and Hudson River Railroad. On March 1st, 1890, Mr. Voorhees was made General Superintendent of this road, and, in 1891, was also appointed General Superintendent of the Rome, Watertown and Ogdensburg Railroad Company. While General Superintendent of the New York Central and Hudson River Railroad he saw his road adopt the controlled manual block signal system on its main line from New York City to Buffalo, N. Y. This was one of the most radical advances in railroad signal practice up to that time. At about the same time (1891), the New York Central put on the Empire State Express, with the phenomenal run of 440 miles in 8 hours, or 55 miles an hour. Mr. Voorhees was the operating head of this achievement, which advertised the New York Central all over the world.

* Memoir prepared by Samuel T. Wagner, M. Am. Soc. C. E.

Largely on account of his combined ability as a railroad engineer and operating official, he was made Vice-President of the Philadelphia and Reading Railroad on February 1st, 1893.

He was called to his new duties when but 46 years of age, and at a most critical period of the railroad's history. Emerging from a state of financial depression, after having passed through several receiverships, the road was in a condition of physical disability. So bad were the conditions financially that, in 1896, the property was sold under foreclosure proceedings. Mr. Voorhees was the operating head during these troublesome times, and was one of the foremost factors in the development of the property of the company. This development was entirely intensive, as the following figures will show:

	1893.	1915.
Mileage	1 170	1 120
Revenue ton-mileage	1 739 000 000	3 141 026 000
Passenger mileage	240 488 000	360 468 000
Earnings	$22 986 000	$46 715 000

These figures show a great development in the freight service, but Mr. Voorhees was also instrumental in perfecting the passenger service, and this will always remain as a monument to him. On May 8th, 1914, he was elected President of the road, succeeding the late George F. Baer.

He was also President of the following subsidiary railroads: Philadelphia, Newtown and New York; Philadelphia and Reading Terminal; Philadelphia and Chester Valley; Philadelphia, Harrisburg and Pittsburgh; Tamaqua, Hazelton and Northern; Reading, Marietta and Hanover; Dauphin and Berks; Philadelphia and Frankford; Schuylkill and Lehigh; and the Williams Valley.

He was a Trustee of his Alma Mater, the Rensselaer Polytechnic Institute, a member of the Holland Society of New York, the Century Club of New York, and St. Nicholas Society of New York; the Huntington Valley Country Club, Racquet Club, Philadelphia Club, and Automobile Club of Philadelphia. He was also a member of Psi Upsilon Fraternity.

One of the honors conferred on Mr. Voorhees was his election to the Vice-Presidency of the American Railway Association in 1904. He was also a Director of the Market Street National Bank of Philadelphia.

He was married, on September 19th, 1871, to Sarah V. Gould, daughter of Judge George Gould, of Troy, N. Y. She died on August 7th, 1872, and on February 4th, 1874, in Syracuse, N. Y., he married Miss Mary E. Chittenden, and is survived by a brother and by his widow, five sons, and three daughters.

It has been said of him:

"Theodore Voorhees belonged to a good school of railroad men. He had a conscience and he had ability. Regarded by many as cold and unapproachable, he was, nevertheless, extremely well qualified to solve all those larger transportation problems which in recent years have vexed the public mind. The dead President of the Philadelphia and Reading Railway was one of these, and an important one, who helped raise the Company from a condition of financial weakness into a state of affluence. By doing that he, with his associates, made their railroad of infinitely greater service to the country it serves than it had ever been during its years of financial ill luck. Mr. Voorhees knew railroading through and through, knew what it had to do to meet popular support and what it had to do to maintain its own self-respect and ability to serve. Alert and keen, industrious and honest, intelligent and full of courage, he could not fail in this generation of vast business development to rise to one of the conspicuous railroad positions of the world."

Another said:

"As a man, there is much to be said of Mr. Voorhees. Of austere presence and striking personality, he commands the respect and esteem of all who come in contact with him. He is a man of very few words. He has always time to listen, and treats his subordinates with the utmost courtesy and respect. His friends all testify to his loyalty and kindness. Those who have had an insight into his private life have characterised his devotion as extraordinary."

For a number of years Mr. Voorhees was a Vestryman of St. Luke's Protestant Episcopal Church, Philadelphia, in which he felt a great interest.

His funeral was held at his home in Elkins Park on Tuesday, March 14th, 1916. About 400 prominent railroad officers, bankers, and professional men were present. As a mark of respect, all trains throughout the Reading System were brought to a stop for one minute, at 11 A. M., the hour of the funeral services.

Mr. Voorhees was elected a Member of the American Society of Civil Engineers on May 6th, 1885. He served as a Director in 1890.

EDMUND BROWNELL WESTON, M. Am. Soc. C. E.*

DIED DECEMBER 9TH, 1916.

Edmund Brownell Weston, the son of the Hon. Gershom Bradford and Deborah (Brownell) Weston, was born in Duxbury, Mass., on March 25th, 1850. He was descended from a family famous as shipbuilders and shipowners whose vessels were known in every seaport of the world. In 1842, the firm, the E. Weston Company, was rated by Lloyd's, of London, as the greatest shipping concern in the world. Mr. Weston's father was a member of the Convention which nominated John C. Fremont for the Presidency in 1856, and also of that which nominated Abraham Lincoln in 1860. His mother belonged to the Brownell family of Little Compton, R. I.

Mr. Weston was educated in the public schools and at the Partridge Academy of his native town, the Highland Military Academy at Worcester, Mass., and by private tutors.

From 1871 to 1874, he was a student in the office of the late Joel Herbert Shedd, M. Am. Soc. C. E., then Chief Engineer of the Providence, R. I., Water-Works which were being constructed at that time. In 1874, Mr. Weston was appointed Assistant Engineer on that work, and held that position until 1877 when Mr. Shedd resigned. Early in 1877, when Samuel M. Gray, M. Am. Soc. C. E., was elected Chief Engineer and Superintendent of the Water-Works to succeed Mr. Shedd, Mr. Weston was appointed one of his Assistant Engineers, and, in this capacity, had charge of the Water Department until 1897, when he resigned to engage in private practice as a Consulting Engineer.

Mr. Weston had been in poor health for a year, but had only been confined to his bed about two months when his death occurred, from hardening of the arteries, at the Des Brisay Hospital, at Boston, Mass. He is survived by two brothers and one sister, all residing in Boston, Mass.

At the time of his death, Mr. Weston was President of the Jewell Export Filter Company, of Providence, R. I., and was widely known as a hydraulic engineer. He had designed and constructed filtration systems, in connection with water-works, in many cities, both in America and abroad, and had traveled practically over the civilized world studying systems of water filtration wherever he went. Until the war broke out in Europe, in 1914, he had spent every summer abroad.

In addition to his practical work on hydraulics, Mr. Weston had contributed much to the literature on the subject, his best known

* Memoir prepared by the Secretary from information on file at the Society Headquarters.

work being "Friction of Water in Pipes", published in 1873. He had also contributed many discussions and four papers on the subject to the *Transactions* of the Society, namely, "Description of Some Experiments on the Flow of Water Through 2½-Inch Rubber Hose and Nozzles of Various Forms and Sizes, Made in the Providence, R. I., Water-Works. Also Results of Investigations Relating to the Height of Jets of Water" (1884); "Description of Some Experiments Made in the Providence, R. I., Water-Works, to Ascertain the Force of Water in Pipes" (1885); "The Results of Investigations Relative to Formulas for the Flow of Water in Pipe" (1890); and "Test of a Mechanical Filter" (1900).

Mr. Weston was a member of the University Club, of Providence, R. I.; the India House Club, of New York City; the Boston Society of Civil Engineers; the Institution of Civil Engineers of Great Britain; and the Authors and Royal Societies of London, England. He was also a member of Corinthian Lodge, F. and A. M., of Providence, R. I.

The following resolution on the death of Mr. Weston was adopted unanimously by the Directors of the Jewell Export Filter Company:

Resolved: That the Company and the Board of Directors have suffered a great loss in the death of Mr. Edmund B. Weston, so long the President and General Manager of this Company, and its Consulting Engineer, and that the Board take this occasion to express its appreciation of the long and faithful service that Mr. Weston rendered the Company, fully realizing that he at all times gave the Company that valuable service which was the result of his intimate knowledge of the business and affairs of the Company and the matters in which it has been engaged, and further realizing that Mr. Weston rendered this service oftentimes handicapped with the discouragement of physical ailments which would have taxed the determination of men less faithful than he; and it is further

Resolved: That copies of the above resolution be sent to the Executors of Mr. Weston's estate.

Mr. Weston was elected a Member of the American Society of Civil Engineers, on December 6th, 1882.

EDWARD THOMAS WRIGHT, M. Am. Soc. C. E.*

DIED MARCH 29TH, 1917.

Edward Thomas Wright was born at Elgin, Ill., on June 30th, 1851. His parents were Paul R. Wright and Emily Grace (Harvey) Wright. He attended the public school at Geneva, and, later, at Cobden, Ill., where the family moved when he was about nine years of age. After graduating from the public schools, he had a private tutor for a year and then attended the Elgin Academy.

After leaving the Academy, Mr. Wright took up the study of engineering, and, in 1872, entered the office of Messrs. Cleveland and French, prominent engineers of Chicago, Ill., and was employed by them on work at Indianapolis, St. Paul, and Chicago, including the South Chicago Drainage, for the development of an industrial district.

Mr. Wright moved to Los Angeles, Cal., in the latter part of 1874 and established himself as a Civil Engineer and Surveyor, maintaining his office up to the time of his death, on March 29th, 1917. He had served three terms as County Surveyor, and was a hydraulic engineer of ability, having done much effective work in developing underground waters and constructing irrigation systems.

In 1873 Mr Wright was married to Lucy Nicholson, who died in 1899. He leaves two sons, George A. and Charles N., who follow his profession, and a widow, Mrs. Capitola B. (Wenzel) Wright, whom he married in March, 1912.

He was a man highly respected for his sterling worth and honesty; ever ready to help others, and loved by many friends.

Mr. Wright was elected a Member of the American Society of Civil Engineers on February 3d, 1886.

* Memoir prepared by W. D. Larrabee, M. Am. Soc. C. E.

AUGUSTUS WATEROUS AGNEW, Assoc. M. Am. Soc. C. E.*

DIED SEPTEMBER 17TH, 1916.

Augustus Waterous Agnew, the only son of William Agnew, now of Victoria, B. C., was born at Montreal, Que., Canada, on June 6th, 1884. He received his engineering education at the Royal Military College, Kingston, Ont., Canada, and at the Crystal Palace Practical School of Engineering, London, England.

Returning to Canada, Mr. Agnew began his professional career, in January, 1906, on preliminary and location surveys for the Canadian Pacific Railway, with headquarters at Montreal, Que.

In July, 1906, he went to Saskatchewan as Instrumentman on the construction of the Grand Trunk Pacific Railway. In December of the same year, he went to Prince Rupert, B. C., where he was employed, as Resident Engineer, on hydrographic, topographical, and town-site surveys, and also on sewer construction, under James H. Bacon, M. Am. Soc. C. E., then Harbor Engineer, Grand Trunk Pacific Railway.

In May, 1909, Mr. Agnew left the employ of the Grand Trunk Pacific Railway to engage in private practice as a member of the firm of Ritchie, Agnew, and Company, with offices at Prince Rupert, B. C. Among other work, this firm had charge of the construction of the first water supply system for the City of Prince Rupert, including three dams, and 14 miles of supply and distribution mains. The firm was also engaged on wharf design and construction; the supervision of surveys, plans, stream measurements, and estimates for a permanent municipal water supply and power project; investigation surveys, etc., for power projects for the Prince Rupert Hydro-Electric Company, Limited, as well as the design and supervision of the subdivision of a town site of 1 000 acres, including topographical and hydrographic surveys and harbor layout, adjoining Prince Rupert, with R. H. Thomson, M. Am. Soc. C. E., acting as Consulting Engineer, and numerous small sewerage and water supply schemes, and water-power investigations.

In 1913, Mr. Agnew became interested in the development of a large water-power at Stamps Falls, near Alberni, Vancouver Island, capable of developing 75 000 h. p., and prepared extensive plans, the development of which, owing to the outbreak of the war, had to be postponed. Mr. Agnew's valuable work on this project will no doubt later bear fruit.

* Memoir prepared by the Secretary from information on file at the Society Headquarters.

In the spring of 1914, he moved to Victoria, B. C., where he took the contract for laying the sewer system of Esquimalt which he completed.

When the 48th Battalion, Canadians, was mobilized there for service in the European war, he transferred from the Corps of Guides to that Battalion, and went overseas as its Captain and Adjutant. The Battalion, through the efforts of Captain Agnew and a few of its officers, went to the front as a unit and an Engineering Battalion of the 3d Canadian Division, and was afterward known as the 3d Canadian Pioneer Battalion. Then it was that Captain Agnew's engineering training came into service, and he made a reputation for it that became known among many of the other Canadian and British Battalions. In April, 1916, after six weeks in Belgium, constructing trenches in the Ypres salient, Captain Agnew was seriously wounded in the head and right arm by high explosive shrapnel, and subsequently suffered the loss of his left eye. After a month in the hospital, he went to Hythe, in Kent, England, where, during his sick leave, he voluntarily gave much needed instruction to non-commissioned officers and men of the Pioneers Depot at South Cæsar's Camp, Shorncliffe, England. He was offered a position as Major on the 3d Canadian Divisional Staff, but as he was eager to get back with his men, he obtained shortened leave and, on August 1st, took command once more of his old Company at the front. After six weeks he was again dangerously wounded while preparing the trenches for the infantry to hold during the battle of Courcelette, on September 15th, and died after an operation, at a Casualty Clearing Hospital near Amiens, France, on September 17th, 1916.

Captain Agnew will be greatly missed by all who knew him well enough to appreciate his many fine personal qualities as well as his ability as an engineer. He was well liked by his fellow officers and by the men whom he had trained and with whom he had fought. The splendid state of organization and efficiency reached by his Battalion was due largely to his tireless energy and ability in engineering and leadership.

He was an Associate Member of the Canadian Society of Civil Engineers, and an Associate Member of the Society of Engineers (Incorporated), of London, England.

Captain Agnew was elected an Associate Member of the American Society of Civil Engineers on July 2d, 1913.

WILLIAM FREDERICK ALFRED ANSON, Assoc. M. Am. Soc. C. E.*

DIED JULY 17TH, 1916.

William Frederick Alfred Anson, the son of the Rev. Alfred W. Anson, was born at Staunton, Va., on August 7th, 1878. He received his early education under governesses and tutors in his home and in England. In 1893, he entered the Episcopal High School, at Alexandria, Va., where he remained until 1896. He then entered Roanoke College, at Salem, Va., where he remained for one year, after which he entered the Virginia Military Institute, at Lexington, Va. He attended the Institute for three years, but was unable to complete his course on account of ill-health. After leaving school, he remained on his father's farm until 1902.

In July, 1902 Mr. Anson entered the employ of the Norfolk and Western Railway Company, as Instrumentman and Inspector on construction work. He remained with this Company until 1907. He then went to Martinsville, Va., where he was engaged in private surveying until 1908.

He was then appointed Inspector in Campbell County under the Virginia State Highway Commission, and, later, was promoted to Resident Engineer by the Commission, with headquarters at Pulaski, Va., having charge of the work in several counties. In April, 1914, he was again promoted by the Commission to be County Engineer of Russell County, with headquarters at Lebanon, Va.

For several years before his death Mr. Anson had been in poor health, and in January, 1916, he went to Richmond where he was examined by a specialist who pronounced his trouble to be chronic appendicitis. He returned to his home in Lebanon, but soon decided to undergo an operation, and, in February, 1916, was operated on at the Bluefield Sanatorium, Bluefield, W. Va. He never fully recovered from this operation, although he returned to his work which he carried on until about a week before his death.

In his professional career Mr. Anson developed those qualities which insure success. He had been connected with the highway work in Russell County only a short time, but he was highly thought of by all the citizens, and the County road authorities were more than satisfied with his management.

In October, 1915, Mr. Anson was married to Miss Vera Seay, of Eagle Rock, Va. He is survived by his widow, his father, brother, and nine sisters.

Mr. Anson was regarded as an engineer of rare ability, and his work with the Virginia State Highway Commission has always been

* Memoir prepared by A. H. Pettigrew, Esq.

highly commended. He was a man of high principles and of strong convictions, never hesitating to stand up for what he thought to be right or to oppose that which he considered wrong. He was a devoted member of the Protestant Episcopal Church, which he had joined in his youth, and he will be greatly missed by his family and his friends and associates.

Mr. Anson was elected an Associate Member of the American Society of Civil Engineers on October 1st, 1912.

ROBERT HAMMOND BOYNTON, Assoc. M. Am. Soc. C. E.*

DIED SEPTEMBER 18TH, 1916.

Robert Hammond Boynton, the son of the late Henry P. and Emma (Hammond) Boynton, was born at Batavia, N. Y., on May 21st, 1886. He received his early education in the public schools of his home town, having been graduated from the High School in 1903. In September, 1904, he entered the Civil Engineering Department of the University of Michigan, where he remained until June, 1906, when he left college to enter the employ of the New York Central and Hudson River Railroad Company, as Chainman on grade reduction between Syracuse and Rochester, N. Y. In April, 1907, he was advanced to the position of Rodman on the same work, but resigned in October, 1907, to return to the University of Michigan, from which he was graduated in June, 1910, with the degree of Bachelor of Science in Civil Engineering.

On July 1st, 1910, Mr. Boynton entered the employ of the Missouri Pacific Railroad Company, as Rodman in the Maintenance of Way Department, with headquarters at St. Louis, Mo., being in charge of track renewal and engaged in general office and field work connected with the Department. In June, 1911, he was transferred to the Construction Department, with headquarters at Marianna, Ark., as Inspector of bridge construction on the Marianna Cut-off. In March, 1912, he was again transferred to the Illinois Division, with headquarters at Illmo, Mo., as Assistant Engineer engaged in routine office and field work connected with the Maintenance Department.

In December, 1912, Mr. Boynton entered the service of the Mississippi River Commission as Recorder on a river survey between Fort Jackson and Port Eads, La.

On April 1st, 1913, he was appointed City Engineer of Frankfort, Ind., by the Mayor, Dr. O. W. Edwards, to fill an unexpired term. In January, 1914, Mr. Boynton was re-appointed to the same office by the newly elected Mayor, Dr. Oliver Gard, and remained in that position until his death, on September 18th, 1916, following a long illness resulting from heart trouble and a complication of diseases, due to exposure and malaria contracted while he was engaged on engineering work in the Southwestern States.

Mr. Boynton was considered one of the best City Engineers that Frankfort, Ind., ever had. He had charge of the design and construction of all street and sewer improvements, and through his efforts a complete set of street and sewer records of the city was made, thereby giving the Municipal Government its first check on sewers, streets,

* Memoir prepared by the Secretary from information on file at the Society Headquarters.

and property owners. He had also completed a map of the City of Frankfort, which has been widely used by those interested in its public improvements.

Mr. Boynton had a genial disposition and readily made and held as friends all who met him. He was most efficient and honest, and through his conscientious efforts and steady application to duty, he received and maintained for his Department perfect work from the contractors employed thereby. He was an expert on filing systems, and at the time of his death was preparing himself for the work of a Consulting Engineer and City Manager.

Mayor Oliver Gard writes of him, as follows:

"Our city was deeply shocked when, on September 18th, the death of Robert H. Boynton, City Engineer, was announced. While he had not been a resident of this city for many years, yet we had learned to regard him highly. His work as City Engineer had been very laborious, but always satisfactory. His efficiency was never questioned. We regret his untimely death."

Mr. Boynton was married on October 15th, 1912, to Miss Zua Rice, a daughter of Mr. and Mrs. John A. Rice, of Frankfort, Ind., who, with his mother, Mrs. Emma Boynton, and a brother, R. Rae Boynton, of Rochester, N. Y., survives him.

He was a member of the Presbyterian Church of Frankfort, Ind., and of Masonic Lodge No. 475, of Batavia, N. Y. He was also a member of the American Society of Municipal Improvements, the Municipal League of Indiana, the Akhenaton Society of Ann Arbor, Mich., the Alumni Association of the University of Michigan, and the Chamber of Commerce of Frankfort, Ind.

Mr. Boynton was elected a Junior of the American Society of Civil Engineers on May 6th, 1914, and an Associate Member on April 18th, 1916.

FRANS ENGSTRÖM, Assoc. M. Am. Soc. C. E.*

DIED MARCH 20TH, 1916.

Frans Engström, the son of Ludwig and Frederika Engström, was born in Stockholm, Sweden, on November 28th, 1851. He was educated in his native city and was graduated from the Latin College and the Royal Polytechnical High School.

After his graduation, Mr. Engström was engaged, for two years, in an architect's office on private surveys, and also as Assistant Resident Engineer on the construction of the Government Railway of Sweden.

In 1881, he came to the United States and settled in Pittsburgh, Pa., where he was engaged with the Pennsylvania Railroad Company, as Draftsman, Chief Draftsman, and Assistant Engineer, until 1895. During this time, he designed and superintended the construction of the Penn Street Freight Station in Pittsburgh, and made plans and specifications for extensive dock construction at Erie, Pa., and at Ashtabula and Cleveland, Ohio. He also made the plans and specifications for all heavy bridge masonry and retaining walls on the Company's lines and for elevating its tracks through Pittsburgh and Allegheny.

In 1895, Mr. Engström was appointed Assistant to the Director of Public Works of the City of Pittsburgh, which position he resigned in 1899. While serving in this capacity, he built the large twin reservoirs at Highland Park, for the Pittsburgh water supply.

During 1899 and 1900, he was engaged in private practice as Civil and Hydraulic Engineer, with an office in Pittsburgh. In 1901, Mr. Engström was appointed Civil Engineer with the Carnegie Steel Company, retaining that position until 1903, when he entered the service of James Stewart and Company as Engineer of Inspection.

In 1905, Mr. Engström went to the American Bridge Company, at Ambridge, Pa., where he remained, as Engineer, until March, 1907. In April, 1907, he removed to Altoona, Pa., where he had been appointed City Engineer, retaining that office until 1915. As City Engineer Mr. Engström had charge of extensive paving work, supervised the planning of the East Altoona disposal plant, built the Seventh Street Bridge, and had charge of other important municipal improvements.

In 1915, Mr. Engström returned to Pittsburgh, but his health having failed, he was not able to engage actively in his profession. He had only been discharged from the hospital where he had been

* Memoir prepared by the Secretary from information on file at the Society Headquarters.

undergoing treatment for five months, when his sudden death from heart failure occurred on March 20th, 1916.

Mr. Engström was a talented and highly educated man. He spoke seven languages, and, being a great reader, was well informed, not only regarding his Profession, but on all subjects of general interest. Among his professional friends, he was considered one of the best estimating engineers in Pennsylvania.

On August 25th, 1884, Mr. Engström was married to Miss Bertha Lindstrum, who, with four children, survives him.

He was a member of the Engineers' Society of Western Pennsylvania, Duquesne Commandery No. 72, Knights Templar, and Syria Temple of Shriners, of Pittsburgh, and of Altoona Lodge No. 102, Benevolent Protective Order of Elks.

Mr. Engström was elected an Associate Member of the American Society of Civil Engineers on May 4th, 1892.

HOWARD ARNOLD GREENE, Assoc. M. Am. Soc. C. E.*

DIED FEBRUARY 2D, 1917.

Howard Arnold Greene was born at Providence, R. I., on September 15th, 1860.

Immediately after his graduation from Brown University, in June, 1884, Mr. Greene entered the employ of S. B. Cushing and Company, Engineers and Surveyors, and remained with that firm until April, 1887. While with this Company, Mr. Greene was engaged on the construction of the Diamond Hill Reservoir for the Pawtucket, R. I., Water-Works, on the greater part of the line of the Union Street Railroad Company, and, as Assistant Engineer, on all the preliminary work for the Providence Cable Railroad Company and the improvements of the Park Land Company, at Elmwood, R. I.

In April, 1887, Mr. Greene was appointed Division Engineer with the Hudson Connecting Railroad Company, under the late P. P. Dickinson, M. Am. Soc. C. E., as Chief Engineer, on the Poughkeepsie Bridge route. When this was finished, in September, 1888, he accepted a position as Engineer for J. W. Coffin and Company, Contractors for the Brigantine Beach Railroad, which position he retained until August, 1889, when he went to the Dunderbergh Spiral Railroad Company, as Division Engineer, until work on that line was stopped.

In December, 1889, Mr. Greene was appointed Locating Engineer with the Danville and East Tennessee Railroad Company. He remained in that position until May, 1890, when he entered the employ of Morris and Darly, Engineers and Contractors, of Bristol, Tenn., as Assistant Engineer on surveys for the State of Virginia and on the erection of a blast furnace at Bristol, Tenn.

In December, 1890, Mr. Greene was engaged as Engineer in charge of the construction of the terminal pier at Atlantic Highlands, N. J., for the Central Railroad of New Jersey. He afterward went to Connecticut as Assistant Engineer on the location of about 30 miles of railroad, under the late Joseph Norton Greene, M. Am. Soc. C. E., as Chief Engineer.

In November, 1892, Mr. Greene entered the employ of the New Jersey Steel and Iron Company as Assistant Engineer. He was promoted, until at the time of the incorporation of the American Bridge Company, he was in charge of erection in the New York District.

In July, 1901, he was appointed Erection Manager of the Pittsburgh Division for the American Bridge Company, which position he

* Memoir prepared by the Secretary from information on file at the Society Headquarters.

held at the time of his death. While serving in this capacity, Mr. Greene had charge of the erection of many important structures and had invented a number of important erection devices which are patented.

He died of pneumonia at his home in Pittsburgh, Pa., on February 2d, 1917, after a brief illness, and is survived by his widow.

Mr. Greene was elected an Associate Member of the American Society of Civil Engineers, on June 5th, 1895.

WILLIAM HERBERT HYDE, Assoc. M. Am. Soc. C. E.*

DIED APRIL 15TH, 1917.

William Herbert Hyde, the son of Peter Lowe and Anne Elizabeth (Copcutt) Hyde, and the grandson of the late John Copcutt, of Yonkers, N. Y., was born at the latter place on December 15th, 1874. He was of English descent, one great-grandfather having been an officer (Captain) in the Revolution and another in the War of 1812. Both his grandfather and his father were engaged in the mahogany and hardwood export and import business in New York City.

Mr. Hyde received his education at the public and grammar schools, and the Yonkers High School, and, in 1895, entered the employ of the New York Central and Hudson River Railroad as Chainman in the Chief Engineer's Department, advancing to Assistant Engineer of the Maintenance of Way Department in 1898. From that time until 1901 he served on location and construction with the Erie Transit Company and the Panama Railroad; on topographical work and estimates for the Barge Canal, New York State; and as Assistant Engineer in the Atlantic City, N. J., Water Department.

During 1901 Mr. Hyde was engaged in surveying and contracting work, being associated with Pittsburgh interests. In April, 1902, he entered the contracting field, forming a partnership with his father-in-law, Mr. W. F. Patterson, of Pittsburgh, Pa., and specialized in shaft and underground improvement work.

During the last ten years, he had conducted his business alone, his operations including coal shafts in Pennsylvania, West Virginia, Nova Scotia, and Alberta, Canada. He built many fine concrete-lined pumping stations underground and successfully completed contracts for such firms as the New River Company, The Berwind-White Company, The Ragland Coal Company, and others of equal standing. He often contributed articles of merit to leading engineering and mining journals, and has been freely quoted as an authority in works on coal mine engineering.

Mr. Hyde died at Scarboro, W. Va., on Sunday, April 15th, 1917, from a bullet wound received while attempting to place an unruly workman under arrest.

He was married on February 15th, 1903, to Miss C. Virginia, daughter of Walter F. Patterson, of Pittsburgh, Pa., who, with four young children, survives him.

Mr. Hyde was a member of the Sons of the Revolution. He was elected a Junior of the American Society of Civil Engineers on April 30th, 1901, and an Associate Member on June 4th, 1902.

* Memoir prepared by S. M. Van Loan, M. Am. Soc. C. E.

JOSÉ PETRONIO KATIGBAK, Assoc. M. Am. Soc. C. E.*

DIED MAY 16TH, 1916.

José Petronio Katigbak was born at Lipa, Batangas, Philippine Islands, on October 4th, 1879. His parents, Mariano Katigbak and Isabel (Macarandang) Katigbak, both of Lipa, Batangas, were wealthy hacienderos, and belonged to the most prominent families in the Province.

His preliminary education was obtained in the Ateneo Municipal de Manila, a school conducted by the Jesuit Order, from which he was graduated, with the degree of B. A., in 1896, at the head of his class.

He then studied medicine for one year in the University of Sto. Tomas, Manila, but this not being to his liking, he decided to study engineering, and while preparing himself to go abroad, he taught Spanish literature in the High School of his native town during 1898 and 1899, his services being given free for the benefit of his countrymen.

Mr. Katigbak then went to Europe and traveled extensively through France, Italy, Germany, and England, and, in 1900, entered King's College of the University of London, from which he was graduated in June, 1903, with the degree of Qualified Civil Engineer and the Certificate of Distinction.

He then entered the Lawrence Scientific School of Harvard University and was graduated with the degree of S. B. in Engineering, in June, 1904.

Subsequent to his graduation, he took the 1904 Special Summer Course at Harvard, specializing in Plane, Railroad, and Geodetic Surveying. On finishing this course, Mr. Katigbak entered the employ of S. D. Warren and Company, of Westbrook, Mass., remaining with that company for 7 months in the capacity of Draftsman, Surveyor and Designer.

In 1905, he returned to his native land and obtained a position with the Bureau of Public Works, where he was employed for about 4 months as Transitman, laying out the City of Baguio in accordance with the plans prepared by G. E. Burnham, Assoc. M. Am. Soc. C. E.

On February 5th, 1906, Mr. Katigbak was transferred to the Department of Engineering and Public Works of the City of Manila, and remained in the service of the City until his death. His promotion was rapid, as may be seen from his service record: February 5th, 1906, Surveyor (temporary); July 1st, 1906, Transitman (probationary); April 1st, 1907, Assistant Engineer; May 1st, 1908, Chief Surveyor; June 1st, 1910, Second Assistant City Engineer; June 3d, 1910, Instruc-

* Memoir prepared by A. Gideon, M. Am. Soc. C. E.

tor in Graphics, University of the Philippines, in addition to his other duties; October 1st, 1911, Superintendent of Streets and Bridges; March 4th, 1914, First Assistant City Engineer.

On several occasions, during the absence of the City Engineer, Mr. Katigbak was Acting City Engineer, and, as such, Acting Member of the Municipal Board. He was a faithful, conscientious, and hard worker, although he was not very robust. The continuous strain of his official duties undermined his health, and, in May, 1916, he left Manila on a short vacation to his home town to recuperate. While there, he contracted typhoid fever. He came back to Manila and had the benefit of the best medical talent, but his health was so undermined that he succumbed to the disease, and died on the morning of May 16th, 1916.

Mr. Katigbak's talents were varied, and he took a great interest in the fine arts, such as painting, poetry, etc. His paintings, while still a student in the Ateneo de Manila, now decorate the walls in several Government offices.

He was of a very happy disposition, and had many friends both in and out of official life, and his death was mourned by the whole population of the Philippine Islands. On the day of his funeral, all flags in Manila were half-masted, and all the City offices were closed. More than 20 000 people were in the funeral cortège. Among the honorary pall-bearers, were the Mayor and the Municipal Board, the chiefs of all the City offices, Justices of the Supreme Court, and many prominent men in civil life. The route of the procession for some miles was decorated with palms and thronged with sympathizers.

The following bodies, organizations, and offices marched in the funeral procession: Department of Engineering and Public Works; the Municipal Board of Manila; Local Members of the American Society of Civil Engineers; The Society of Engineers and Architects of the Philippines; The Columbian Association; the Bureau of Public Works, and many other organizations.

The Municipal Board, by resolution, set aside a plot for the deceased in the area reserved for the burial of illustrious and heroic dead, and, in addition, adopted the following resolution of condolence:

"Excerpt from the Minutes of the Municipal Board of May 16, 1916.

"*Whereas,* the Almighty in His Infinite wisdom has seen fit to remove from our midst our most esteemed friend and former public official, Mr. José P. Katigbak, who departed this life Tuesday, May 16th, 1916; and

"*Whereas,* it is the desire of this Body to express its condolence to the family, friends, and relatives of the deceased and its appreciation to the public generally of the efficient manner in which his services to the City of Manila as a public official had always been performed; now, therefore, be it

"Resolved: That the sincere sympathy and condolence of the Municipal Board be, and the same hereby are, extended to the immediate family, relatives, and friends of the late José P. Katigbak and that an expression of commiseration be tendered to them in this, their hour of bereavement; and be it further

"Resolved: That this Body do, and it hereby does, publicly express its keen appreciation for the manifold labors performed by him as a City official; for the betterment of conditions in the City of Manila; and be it further

"Resolved: That the flag at the City Hall building be displayed at half-mast until the funeral day, inclusive, on which date the flags on all City buildings shall be similarly displayed; and be it further

"Resolved: That a certified copy of these resolutions be sent to the widow and father of the deceased and to His Excellency, the Governor-General; and be it further

"Resolved: That this Board attend in a body the funeral ceremonies to be held over the remains of our departed friend."

At a later session of the Board, it was decided to name one of the principal streets, facing the New Luneta, Katigbak Boulevard.

Mr. Katigbak was married, in 1910, to Miss Trinidad Buenaventura, who, together with his father, two brothers, and one sister, survives him. He was very fond of children and his greatest regret was that he had no issue.

Mr. Katigbak was a member of The Institute of Engineers and Architects of the Philippine Islands, of which he was the first President. He was also a member and one of the founders of the Philippine Columbian Association, a society made up of Filipinos who have received college degrees in the United States.

Mr. Katigbak was elected an Associate Member of the American Society of Civil Engineers on April 1st, 1914.

GEORGE GERE MacCRACKEN, Assoc. M. Am. Soc. C. E.*

DIED AUGUST 1ST, 1913.

George Gere MacCracken, the son of Henry Mitchell MacCracken (Chancellor Emeritus of New York University), and Catherine (Hubbard) MacCracken, and a grandson of the Rev. John Steele MacCracken, of Ohio, and the Rev. Thomas Swan Hubbard, of Vermont, was born on March 8th, 1878, at Toledo, Ohio.

He was educated at the Lyons Collegiate Institute, Broadway and 21st Street, New York City, and, in 1894, entered the New York University School of Applied Science, from which he was graduated in 1898 with the degree of B. S., receiving the degree of C. E., in 1899. He was Inman Fellow at New York University during the college year of 1898-99, and also took special courses in electricity at the Columbia University School of Mines. In 1899, he was awarded the Hoe Sanitary Engineering Prize for the best work in sanitary investigation.

In 1899, Mr. MacCracken was appointed to the Engineering Staff of the Manhattan Railway Company. He remained with that Company until 1902, when he became Assistant Engineer of the Interborough Rapid Transit Company, holding that position until 1908. His principal work with these companies was the supervision of the construction of the Elevated Railway Terminal Yards at 159th Street and the Harlem River and at Third Avenue and 179th Street. These yards cost about $500 000 and required very skillful engineering.

In 1908, he organized the Alboro Contracting Company, of which he became President, and, later, the MacCracken, Hauer, Terry Company, engaging in the general contracting business.

Mr. MacCracken was of large and commanding presence and was possessed of a genial nature, although rather exclusive in his choice of associates. His thorough preparation for engineering work, coupled with his industrious and conscientious nature, gave promise of substantial accomplishment.

He lost his life by drowning off Glen Cove, Long Island, on August 1st, 1913, having been stricken by heart failure after the exertion of bringing his launch through a severe storm on Long Island Sound. He was buried in Sleepy Hollow Cemetery, at Tarrytown, N. Y.

Mr. MacCracken was married on April 29th, 1907, to Martha J. Hall, the daughter of the late John H. Hall, who, with one son, Williston Ward MacCracken, born in 1910, survives him.

* Memoir prepared by George H. Pegram, President, Am. Soc. C. E., and Charles H. Snow, M. Am. Soc. C. E.

During his college course, Mr. MacCracken was a member of the Psi Upsilon Fraternity, the Red Dragon Senior Society, and the Chemical Society, and was President of the Engineering Society. Later, he was a Trustee of the American Savings Bank of New York, and a member of the Railroad Club and the Psi Upsilon Club.

Mr. MacCracken was elected a Junior of the American Society of Civil Engineers on February 5th, 1901, and an Associate Member on February 3d, 1904.

STANLEY HASTINGS McMULLEN, Assoc. M. Am. Soc. C. E.*

DIED JULY 12TH, 1916.

Stanley Hastings McMullen, the third son of the late H. D. McMullen, was born at Aurora, Ind., on June 4th, 1872. He received his early education in the public schools of his home town, leaving the High School to enter the United States Naval Academy at Annapolis, Md., in May, 1893. He was obliged, on account of ill health, to leave the Naval Academy during his third year, and in September, 1895, he entered Purdue University where he took the course in Civil Engineering.

In May, 1899, Mr. McMullen entered the employ of the Illinois Central Railroad Company, as Track Apprentice. He remained with this Company until September, 1903, having attained the position of Resident Engineer in charge of construction. He was obliged to go West on account of his health and from September, 1904, to January, 1905, was engaged with the State Engineer of Colorado on the Colorado-Kansas Irrigation suit then before the Courts. On his return East he served as Division Engineer on 14 miles of construction for the Indianapolis and Louisville Traction Line, from April to December, 1906.

In January, 1907, Mr. McMullen went to El Paso, Tex., where he was engaged in private surveying until March of that year, when he was appointed City Engineer of Aurora, Ind. He held that position until October, 1907, during which time he planned the sewer system for that city and many street improvements.

Mr. McMullen then returned to Colorado and served as Assistant Engineer with the Denver Reservoir and Irrigation Company until January, 1908, when he returned to Anderson, Ind., to engage in the private practice of engineering.

In September, 1913, he accepted a position as Assistant City Engineer and Draftsman at El Paso, Tex., which he held until April, 1915. Mr. McMullen afterward devoted his time to private practice in and around El Paso, and also at Lake City, Fla., until May, 1916, when he returned to Jeffersonville, Ind., fully intending within a short time to return to Orlando, Fla., where he had accepted a position. His health had been failing for some time and at the urgent request of his family he entered Christ Hospital, at Cincinnati, Ohio, where he died after an operation undertaken in a last effort to save his life.

In 1901 he was married to Miss Laura Frank, of Jeffersonville, Ind., who, with his three brothers, survives him.

* Memoir prepared by the Secretary from information on file at the Society Headquarters.

Mr. McMullen was regarded as an engineer of exceptional ability and his work was always highly commended by his superiors. Owing to the fact that he was obliged, on account of his health, to live and work in many different parts of the country, his experience covered almost every phase of engineering work. He was a man of indomitable will power and of strong convictions, never hesitating to stand up for that which he deemed to be right or to oppose that which he considered wrong. He was a loyal and devoted member of the Methodist Episcopal Church which he had joined in his youth, and he will be greatly missed by his family, his friends, and associates.

Mr. McMullen was a member of the Indiana Engineering Society, and was elected an Associate Member of the American Society of Civil Engineers on November 3d, 1915.

PHILIP HENRY PARTHESIUS, Assoc. M. Am. Soc. C. E.*

DIED FEBRUARY 15TH, 1916.

Philip Henry Parthesius, the eldest son of Dorathea Jordan and Julius Parthesius, was born at Troy, N. Y., on December 15th, 1881. He received his early education in the public schools of Troy and, later, attended the Rensselaer Polytechnic Institute, from which he was graduated with high honors in 1904, receiving the degree of Civil Engineer.

Mr. Parthesius began his professional work in the Bridge and Construction Department of the Pennsylvania Steel Company, at Steelton, Pa., remaining there about a year. On leaving this work, he became Assistant Engineer on the construction of a large extension to the water-works system of Troy, N. Y. In this capacity, he was engaged in the building of storage reservoirs, pipe lines, dams, controlling works, etc. On the completion of this work which took about 3 years, Mr. Parthesius became connected with the New York State Civil Service Commission at Albany, in the capacity of Engineering Examiner, and, in October, 1913, he was made Assistant Chief Examiner of this Commission.

While with the Civil Service Commission, he had much to do with the preparation and rating of examinations for engineering positions in all State Departments, including those of the State Engineer and Surveyor, Public Service Commission, and the Highway Department, all of which required a large number of men to handle the extensive work on the Barge Canal, New York City subways, and the improved highway systems.

Mr. Parthesius was a wide reader and a keen student; a man of calm, even temperament, a true friend and gentleman, who possessed the confidence and respect of those who had the pleasure of his acquaintance. His early demise removed from the Profession one who was deeply devoted to his work, and who was just entering the pathway leading to a successful career in engineering work. His sudden illness and death was deeply lamented by a wide circle of friends and associates. He died on February 15th, 1916, after a brief illness.

Mr. Parthesius was unmarried, and is survived by his mother and one brother, Henry J. Parthesius, also a civil engineer.

He was a member of the Albany Society of Civil Engineers and of the Eastern Society of Civil Engineers.

Mr. Parthesius was elected an Associate Member of the American Society of Civil Engineers on November 1st, 1910.

* Memoir prepared by H. O. Schermerhorn, Assoc. M. Am. Soc. C. E., Troy, N. Y.

LEWIS ROBERTS POMEROY, Assoc. Am. Soc. C. E.*

DIED MAY 7TH, 1917.

Lewis Roberts Pomeroy was born at Port Byron, N. Y., on February 5th, 1857, but his early life was spent in Milwaukee, Wis., where his father was in business. In his fifteenth year he came East, and attended school at the Irving Institute in Tarrytown, N. Y., for one year. His subsequent education was obtained entirely by his own efforts, for, from this time, he supported himself by filling various clerical positions until, in 1880, he became Secretary and Treasurer of the Suburban Rapid Transit Company, which position he filled until 1886.

From 1886 to 1895 Mr. Pomeroy was with the Carnegie Steel Company, during which time he introduced basic boiler steel for locomotives and special forgings. He was a painstaking student of locomotives and locomotive details, and exerted a strong and effective influence in improving such materials and in the detailed design of locomotives. Subsequently, he was engaged in the same kind of work jointly for the Cambria Steel Company and the Latrobe Steel Company. From 1899 to 1902, he was Assistant General Manager of the Schenectady Locomotive Works, and from 1902 to 1908, he was representative in the railway field for the General Electric Company.

Engineers owe Mr. Pomeroy a debt of gratitude for his studies in the electrification of steam railroads, for his knowledge and experience enabled him to reveal many important phases of this problem. His earlier work in connection with electric power in railroad and manufacturing shops was of great importance in methods of modernization, and he was undoubtedly among the highest authorities on railroad shop layouts and equipment.

Mr. Pomeroy's next appointment was as Assistant to the President of the Safety Car Heating and Lighting Company, and, subsequently, he was appointed Chief Engineer of the Railway and Industrial Divisions of J. G. White and Company. In June, 1914, he was appointed Manager of the New York Sales Office of the United States Light and Heating Company, but later opened an office as Consulting Engineer in New York City.

Mr. Pomeroy had been in the railway and railway supply business for more than thirty-five years, and many officials and engineers, especially those connected with railroads and railroad developments, will sadly miss his friendly helpfulness. His own wide engineering knowledge came through painstaking study, experience, and contact.

* Memoir prepared by George M. Basford, Esq., New York City.

His kindly nature was attracted to young men who were struggling to succeed; his natural desire was to encourage and aid them, and to lead them to improve and to take important places in the world's work. Would that we had more men inspired as he was with the desire to help others! It became an important part of Mr. Pomeroy's life-work to discover young engineers of ability and to bring them to the front, and this will be to him an enduring monument.

His death, which was very sudden, occurred at his home in East Orange, N. J., on May 7th, 1917. He is survived by his wife and two children.

Mr. Pomeroy was elected an Associate of the American Society of Civil Engineers on April 2d, 1890.

WILLIAM THOMAS SHAW, Assoc. M. Am. Soc. C. E.*

DIED FEBRUARY 26TH, 1916.

William Thomas Shaw, a son of Ebenezer A. and Betsey S. (Dunham) Shaw, was born in Middleborough, Mass., on May 6th, 1880. His father was a veteran of the Civil War.

From the local schools, Mr. Shaw went to Dartmouth College where, as at home, he was a renowned exponent of the National game, filling the positions of pitcher and first baseman on the 'Varsity nine. He was a member of the Phi Delta Theta fraternity, and of the Senior Society, Casque and Gauntlet.

After receiving the degree of B. S., in 1904, he entered the Thayer School of Civil Engineering, an associated school at Hanover, N. H., founded in 1871 by Gen. Sylvanus Thayer, father of the United States Military Academy. There he was well grounded in the fundamentals of Civil Engineering, under the directorship of that able and scholarly professor, Robert Fletcher, M. Am. Soc. C. E. Mr. Shaw rounded out his education by spending the summer and fall of 1904 in surveying; the spring of 1905 as Aide on the Forestry Survey of Dartmouth College Grant, Coos County, New Hampshire; and from June to November, 1905, as Instrumentman on electric railway construction at Sterling, Mass.

Receiving the degree of C. E., in 1906, Mr. Shaw was, for a short time, in the field office of the Charles River Basin Commission, as Computer. In November, 1906, he went to Pennsylvania, where he was employed by the Cumberland Valley Railroad for two months as Assistant to Crosby Tappan, Construction Engineer. He was then put in charge of 6 miles of revision work to eliminate grade crossings, reduce grades, and double-track.

On May 5th, 1909, Mr. Shaw entered the service of the Boston Elevated Railway Company, under the late George A. Kimball, M. Am. Soc. C. E., Chief Engineer. He was employed in the Cambridge Office of the Bureau of Elevated and Subway Construction, first as Computer and Checker, and, later, as Assistant Engineer during the construction of the Main Street Subway. In the latter position, Mr. Shaw was engaged on the design and plans of the Eliot Square Car-House, Cambridge, including foundations, floor, retaining walls, and track layout. The Car-House is the Cambridge Terminal of the Cambridge Subway, and consists of a large area of storage tracks, both covered and in the open, repair shops, etc., for the large cars used in the Subway.

* Memoir prepared by F. S. Weston, Assoc. M. Am. Soc. C. E.

Mr. Shaw was rapidly advancing in his chosen field when, on July 18th, 1911, just as he was about to sign the plans which he had been preparing, he was forced to resign on account of pulmonary tuberculosis. After seeking to regain his health in various places, he returned to Middleborough, Mass., and built a home adjacent to the place of his birth, where he continued to make a brave, but losing, fight against the ravages of the disease, by working in the open air.

He was a clean young man, ambitious, conscientious, with a strong sense of duty and loyalty; of excellent judgment and clear and orderly way of thinking. He had that happy faculty of making friends wherever he went and also retaining them. To the last, he was hopeful and cheerful, laying his plans for the future and ever ready to help others.

On June 30th, 1910, Mr. Shaw was married to Madeleine R., daughter of William Scott and Mary (Bryant) Fleming, of Greencastle, Pa., who survives him.

He was a member of the Boston Society of Civil Engineers and the Thayer Society of Engineers. He was also a member of the First Congregational Church, the Masons, and the Sons of Veterans.

Mr. Shaw was elected an Associate Member of the American Society of Civil Engineers on June 30th, 1910.

WILLIAM STUART SMITH, Assoc. M. Am. Soc. C. E.*

DIED NOVEMBER 5TH, 1916.

William Stuart Smith was born in Troy, N. Y., on April 15th, 1856. He was the eldest of three children, and was descended on both sides from Revolutionary ancestors. His grandfather, Captain Levi Smith, was a contemporary of Commodore Vanderbilt in the steamboat business, and his father, William Stuart Smith, was long connected with the New York Central Railroad. His mother was Delia Marble Newton, of Bennington, Vt.

He was educated at private schools and the College of Montreal, and after removing with his parents to Rochester, N. Y., in his seventeenth year, he attended the school of Professor George Hale.

Mr. Smith's first engineering work was with a field party during the four-tracking of the New York Central Railroad. On June 1st, 1876, he entered the office of the City Surveyor, of Rochester, and, in 1883, he was made First Assistant.

In 1886, he entered the employ of the Warren-Scharf Asphalt Paving Company, which, in 1900, became The Warren Brothers Company, of Boston, Mass. Mr. Smith was also District Manager of The Warren Chemical Company, of New York City, and carried on a large roofing business. In 1914, he purchased the Western New York rights of The Warren Chemical Company, and conducted the business under the name of The W. Stuart Smith Company. He continued his connection with The Warren Brothers Company, and, at the time of his death, was a Director and Vice-President of that Company. Mr. Smith was frequently consulted and generally recognized as an expert in paving and roofing.

About fifteen years ago he was interested in a plantation operated by The Indian River Pineapple Company.

Mr. Smith was a man of fine address and had a lovable personality. He had hosts of friends whom he entertained at his summer home, Sunny Side, at Pultneyville, N. Y., on Lake Ontario. He was a lover of fine horses, and maintained a dashing span long after the automobile had come into vogue.

He was a member of many professional, social, patriotic, and athletic organizations, a Thirty-second Degree Mason, and a communicant of Christ (Episcopal) Church, of Rochester, N. Y.

On September 4th, 1879, he was married to Minnie Pomeroy Sackett, of Rochester, who survives him. He is also survived by two

*Memoir prepared by John F. Skinner, M. Am. Soc. C. E.

brothers, Charles S. Smith, of Connecticut, and Frederic Levi Smith, of Rochester, two sons, Captain Lawrence N. and Donald Stuart Smith, two daughters, Mrs. Marion Elizabeth Stafford, and Mrs. Carol Content Gard, and four grandchildren.

Mr. Smith was elected an Associate Member of the American Society of Civil Engineers on January 8th, 1902.

SAMUEL FORSYTHE THOMSON, Assoc. M. Am. Soc. C. E.*

DIED JANUARY 30TH, 1917.

Samuel Forsythe Thomson was born in Charleston, S. C., on June 5th, 1872. His parents were Samuel and Elizabeth Craig Forsythe Thomson, natives of County Antrim, near Belfast, Ireland, who came to Charleston at an early age, were married and made their home there. His father was a prominent merchant in Charleston for many years, and was well and favorably known throughout that section of the South, being prominently identified with many of the activities of his community.

Mr. Thomson first attended a public school in Charleston, and, after finishing the course there, entered the Charleston High School, from which he was graduated with honor in June, 1889. A life-long friend of the family testifies to the fact that he was a favorite alike with teachers and pupils. He then attended a school at Boylston and Tremont Streets, Boston, Mass., where he was prepared for a higher education. Mr. Thomson entered the Massachusetts Institute of Technology in September, 1890, and specialized in civil engineering.

During the summer vacations of 1891 and 1892 he was engaged on engineering work with the firms of Aspinwall and Lincoln and E. A. W. Hammatt, Civil Engineers, of Boston, Mass., and during the summer of 1893 attended the summer school of the Institute of Technology at Keeseville, N. Y. In the summer of 1894 he was employed with John W. Arnold, United States Marshal, of Chicago, Ill., and later for a short period was with the late Col. George E. Waring, Jr., in the Department of Street Cleaning, New York City.

During 1895-96 Mr. Thomson took an additional course at the Institute of Technology in railroad engineering. He was graduated from the Institute in June, 1896, with the degree of Bachelor of Science in Civil Engineering.

After graduation he re-entered the service of the Department of Street Cleaning of New York City under the Commissioner, Col. George E. Waring, Jr., and until May, 1898, was a member of his personal staff. During his administration as Commissioner, Col. Waring gained an international reputation by reorganizing the Department, introducing many reforms, and placing it on an efficient and business-like basis. Mr. Thomson acted as Chief Assistant to the Master Mechanic, and had charge under his supervision of the design and erection of two steel storage dumps, also the general design and repairs to city property, such as stables, dumps, and scows, and of snow removal above 59th Street, Manhattan. He also supervised the introduction of the system of separation of city refuse throughout

* Memoir prepared by Robert Ridgway and George A. Taber, Members, Am. Soc. C. E.

the entire city, and of the construction and operation of the first crematory used for the economical utilization of city wastes for the development of power.

From May, 1898, to April, 1903, Mr. Thomson was the Assistant to the Professor of Civil and Military Engineering at the United States Military Academy, West Point, N. Y., and then entered the service of the Commission on Additional Water Supply for the City of New York, this commission having been appointed to investigate all available sources for an additional water supply for the city. Mr. Thomson was in charge of the New York office of the Long Island Department, under Walter E. Spear, M. Am. Soc. C. E., which Department was engaged in the investigation of the available underground supply on Long Island.

On July 23d, 1903, he was appointed an Assistant Engineer in the Engineering Department of the Rapid Transit Railroad Commission, of New York City, and was assigned to the Fifth Division under Robert Ridgway, M. Am. Soc. C. E., Division Engineer, on the construction of the rapid transit tunnels under the East River from South Ferry, Manhattan, to Joralemon Street, Brooklyn, which tunnels are now in operation. He had much to do with the triangulation and other survey work in connection with the alignment of the tunnels, and for a time was in charge of the general office management of the Division. He resigned on March 5th, 1906, to accept an appointment as Assistant Engineer in the service of the Board of Water Supply of the City of New York, of which J. Waldo Smith, M. Am. Soc. C. E., was Chief Engineer, and was assigned to the Wallkill Division of the Northern Aqueduct Department by the Department Engineer, Robert Ridgway. The Wallkill Division, with headquarters at New Paltz, included about 12 miles of Catskill Aqueduct construction, which aqueduct was designed for a daily capacity of 500 000 000 gal. The most notable feature of construction was the deep rock pressure tunnel, 4.4 miles long, under the Wallkill Valley, this tunnel being located at a maximum depth of about 520 ft. below the hydraulic gradient. Other features of construction were the south half of the Bonticou grade tunnel through the Shawangunk Mountains, about 6¾ miles of cut-and-cover aqueduct on the slope of these mountains and on the east side of the Wallkill Valley, and the blow-off chamber and conduit leading from the aqueduct to the Wallkill River. The cost of construction of the work on the Division was approximately $5 500 000. For a time Mr. Thomson was in charge of the organization of the division office, and was instrumental in establishing methods and a routine which proved to be most efficient in carrying out the work. Following the transfer to other work of James F. Sanborn, Assoc. M. Am. Soc. C. E., Division Engineer, Mr. Thomson, on February 1st, 1912, was made Acting Division Engineer in his place, and so remained until the practical completion of the work.

Of his record on the Wallkill Division, Ralph N. Wheeler, Assoc. M. Am. Soc. C. E., now in charge of the Northern Aqueduct Department, writes:

"His work as an organizer of a working force and of systems of routine, records, filing, etc., was unusual. In no other division of this department were records kept so faithfully and well, supplies, equipment, etc., so completely accounted for, and large and small matters relating to the work so completely recorded. I shall always believe that this was due very largely to Mr. Thomson's faithful oversight. His loyalty to his superiors, consideration for his subordinates and in fact his every-day devotion to his work, even in the small and often disagreeable details, are the characteristics for which we shall always remember him."

On June 15th, 1914, he left the service of the Board of Water Supply and accepted employment with the Kingsbridge Contracting Company, of New York City, as Civil Engineer. He was identified with the operations of that company, particularly in the construction of the large outlet sewer which was built in connection with the Seventh Avenue Rapid Transit Subway, from that avenue to the Hudson River, the cost of this sewer being about $500 000. He was also connected with the construction of the large trunk sewer in East 41st Street, as well as with other projects carried out by that company.

On his return home from the excursion during the Annual Meeting of the American Society of Civil Engineers, on January 18th, 1917, Mr. Thomson was stricken with the illness which resulted in his death twelve days later.

A friend, of many years standing, who knew the personal as well as the business side of his character, writes:

"To those who knew him intimately many of his finer qualities were alone revealed. His friendship possessed those priceless and rare characteristics which endeared him to all who were privileged to be counted among his friends. Its endurance was unquestionable, and at all times his active and unqualified support to a friend could always be depended upon. His sense of honesty was also of the highest quality. In all his dealings the moral obligation in a business transaction was as important and binding to him as any of the legal provisions, and this quality characterized all of his acts through life. He was in all respects an ideal friend, in the finer sense of the word, and none could desire or have a better."

Another associate of his in his later work says:

"I consider him one of the finest characters I have ever met. In the three years of our acquaintance and friendship we were together considerably during business hours and out of them. At all times and under all conditions, many of which were often trying, his pleasant smile and cheery way never left him. He was very careful not to hurt any one's feelings, and never spoke disparagingly or unkindly of any

person; he generally had some explanation or excuse for the failings of the one who might be under discussion and who had not measured up to the full stature of a man."

Loyalty to his principles and to his friends, honesty, humanity, and steadfastness of purpose were prominent among the underlying qualities of his nature, and the character built on such a foundation was one which his friends will long remember with affection and pride.

Mr. Thomson was married on September 29th, 1898, to Miss Jennie A. Milton of Danvers, Mass., who with one daughter, Elizabeth, survives him. He was a member of the New England Water Works Association, the Municipal Engineers of the City of New York, and the Brooklyn Engineers Club. He was much interested in the work of the Fourth Unitarian Church, of Brooklyn, N. Y., of which he was a member.

Mr. Thomson was elected an Associate Member of the American Society of Civil Engineers on January 3d, 1906.

GEORGE EDWARD VANSITTART, Assoc. M. Am. Soc. C. E.*

DIED MAY 14TH, 1916.

Major George Edward Vansittart, 13th Battery, 4th Canadian Field Artillery Brigade, died on May 14th, 1916, from shell wounds received while on active service somewhere in France.

The only son of John Pennyfather Vansittart, Public Works Department, India, George Edward Vansittart was born at Mussoorce, India, on October 7th, 1884. He came of a long line of naval and military ancestors, on both sides. His great-grandfather and his uncle on his father's side were Admirals in the British Navy, and Bisham Abbey, in Berkshire, was for many years the home of his great-grandfather, of which branch of the family he was the last young male representative. His great-grandfather on his mother's side was Colonel Alexander Light who commanded with distinction in India, and elsewhere, the 25th Queen's Own Borderers.

George Edward Vansittart came to Canada at an early age, and was a graduate of the Royal Military College of Canada and also of McGill University. At the conclusion of his University career, in 1906, he adopted the profession of Civil Engineering. During the next two years he was employed as Resident Engineer on the construction of the Midland Railway of Manitoba and Assistant Engineer on the Canadian Pacific Railway Irrigation Project, at Calgary, Alberta.

In the spring of 1909, Mr. Vansittart was engaged by Messrs. Smith, Kerry and Chace, Consulting Engineers, of Toronto, Ont., as Assistant Engineer on the construction work of the Crane Falls Power and Irrigation Company, of Boise, Idaho. Before severing his connection with this Company to enlist for active service, he held the position of Vice-President and General Manager.

When the Great War broke out, this gallant officer returned to Canada and enlisted as a subaltern in the Field Artillery, where his great ability gained for him rapid promotion, and, in a short time, he was gazetted as Major and given command of a battery. By the time his battery reached England, it was considered to be the smartest and most efficient unit in the artillery of the division to which it belonged. Decorations and further rapid promotions were prophesied for Major Vansittart, whose untiring energy, able judgment, and sound gunnery had impressed every one who came into contact with his work. A Canadian Staff Officer writing a personal letter said:

* Memoir prepared by F. H. Peters, Assoc. M. Am. Soc. C. E.

"Vansittart is the best battery commander of the outfit, and his gun positions are the pride of the district. Neighboring artillerymen are sent down to see them as a model of what gun positions should be. The enemy have never located them."

Major Vansittart was mentioned in a despatch from General Sir Douglas Haig, published June 15th, 1916, for gallant and distinguished service in the field.

George Edward Vansittart was elected an Associate Member of the American Society of Civil Engineers, on March 5th, 1912. He was also an Associate Member of the Canadian Society of Civil Engineers.

JAMES MADISON WARNER, Assoc. M. Am. Soc. C. E.*

DIED MARCH 5TH, 1916.

James Madison Warner, a son of the late William H. Warner, was born at Syracuse, N. Y., on August 4th, 1882. He was educated in the public schools of that city, and was graduated from the High School. In 1902, he entered the University of Illinois from which he was graduated in June, 1908, with the degree of B. S. in Civil Engineering.

Prior to entering college and during his summer vacations, Mr. Warner had been employed by the Chicago and Western Indiana Railroad Company as Chainman, Rodman, and Instrumentman on the work of track elevation into Chicago, Ill.

In 1908, he accepted a position as Instrumentman with the Grand Trunk Pacific Railroad Company, in connection with the grading and alignment of about 30 miles of track from Winnipeg, Man., west, and in 1909, he was engaged in the same capacity on track revision and grade alignment for the Chicago and Northwestern Railroad Company, at La Crosse, Wis.

Later in the same year, Mr. Warner went to Oralo, Tex., as Resident Engineer on 17 miles of construction on the Pecos and Northern Texas Railroad, a branch of the Santa Fé System. On the completion of this work in 1910, he was transferred to Sweetwater, Tex., where he had charge of the construction of 13 miles of heavy construction, large yards, a 600-ft. dam, etc.

In 1912, he abandoned railroad engineering and returned to Syracuse, N. Y., as Chief Engineer of the Onondaga Litholite Company, of which his brother, Mr. Henry P. Warner, is President. In addition to his duties as Chief Engineer, Mr. Warner became prominently identified with the general management of the business until March, 1915, when he was stricken with appendicitis. An operation followed, but complications developed which caused his death on March 5th, 1916, at the Syracuse Hospital for Women and Children.

While at college Mr. Warner had been prominent in student affairs, and was regarded as one of the most popular members of the undergraduate body. In his professional and business career, he had developed those qualities which insure success, and, despite his comparative youth, he had already won for himself a leading place in the industrial activities of Syracuse. His manly qualities of mind and heart had won for him many friends, and his death was a severe loss to them and to his family. Mr. Warner is survived by two brothers,

* Memoir prepared by the Secretary, from information on file at the Society Headquarters.

Mr. Henry P. Warner, of Syracuse, N. Y., and Mr. Robert K. Warner, of Ouray, Colo., and by two sisters, Mrs. Henry K. Chadwick and Mrs. George A. Hanford, of Syracuse, N. Y.

He was a member of the Delta Tau Delta Fraternity, University of Illinois Chapter, the Citizens, Onondaga Golf and Country, and the University Clubs, of Syracuse, N. Y., and Champaign Lodge, B. P. O. E.

Mr. Warner was elected a Junior of the American Society of Civil Engineers on April 6th, 1909, and an Associate Member on December 3d, 1913.

WILLIAM COOPER CUNTZ, Assoc. Am. Soc. C. E.*

DIED NOVEMBER 2D, 1916.

William Cooper Cuntz, the son of the late Emil A. H. Cuntz and Frances Cooper Cuntz, was born at Hoboken, N. J., on January 21st, 1871. His maternal grandfather, William Cooper, after whom he was named, was a well-known naturalist and one of the founders of the New York Academy of Science. His maternal grandmother, Mary Wilson, was of New England ancestry, going back to the days of the Pilgrims, and including officers and soldiers in the Revolutionary Army.

Mr. Cuntz was educated at the Hoboken Academy and at Stevens Institute of Technology, having been graduated from the latter in 1892 with the degree of Mechanical Engineer. He then became connected with the Pennsylvania Steel Company, of Steelton, Pa., first with the Bridge and Construction Department, which he represented as Resident Engineer in Boston, Mass., and as European Resident Engineer, in London, England. In these capacities he rendered many important services to his Company, notably in connection with the erection of the North and South Stations in Boston, and in securing the contract for the construction of the North German Lloyd piers, in Hoboken, N. J. He afterward entered the Sales Department, serving as Assistant General Manager of Sales, in Philadelphia, Pa., and, later, as District Sales Manager, in Steelton, Pa.

At the outbreak of the Spanish-American War in 1898, Mr. Cuntz volunteered for service in the Artillery. In 1910, he was appointed by President Taft as a delegate to the International Railway Congress held at Berne, Switzerland, the present Secretary of the Interior, the Hon. Franklin K. Lane, being the Chairman of the American Delegation.

In the same year (1910), Mr. Cuntz severed his connection with the Pennsylvania Steel Company to become a Director and the General Manager of the Goldschmidt Thermit Company, of New York City, which position he held until his death which occurred on November 2d, 1916, at Auburndale, Mass., where he had gone on a visit for the benefit of his health which had been impaired by an operation for appendicitis a year before. Under his management the business of this Company was not only increased greatly, but equal progress was made in the technical development of the Thermit process. His wide experience in the steel industry naturally caused him to take a particular interest in the manufacture of carbon-free metals, and it is due

* Memoir prepared by W. R. Hulbert, Esq.

largely to his perseverance and initiative that these metals and alloys are coming into such general use.

He is survived by his widow, who was Miss Katherine Hughes Edwards, of Hagerstown, Md., a descendant of Jonathan Edwards, and by his two children, William Cooper, Jr., and Emil Edwards. He is also survived by two brothers, John H. Cuntz, of Hoboken, N. J., and Hermann F. Cuntz, of New York City.

Mr. Cuntz was a member of the following clubs and societies: American Iron and Steel Institute, Engineers Society of Pennsylvania, Engineers Club of New York, Railroad Club of New York, Engineers Club of Boston, University Club of Philadelphia, Chemists Club of New York, Boston Athletic Association, India House, Transportation Club, Harrisburg Club, American Electric Railway Association, New England Iron League, West Side Tennis Club, Richmond Hill Association, Kew Gardens Civic Association, Kew Gardens Country Club, Oak Island Yacht Club, and the Conococheague Club of Hagerstown, Md.

Mr. Cuntz was elected an Associate of the American Society of Civil Engineers, on September 6th, 1910.

GEORGE HENRY FROST, Assoc. Am. Soc. C. E.*

DIED MARCH 15TH, 1917.

George Henry Frost, the son of Ebenezer and Caroline (Harwood) Frost, was born on July 9th, 1838, at West Hawkesbury, Ont., Canada, where his father was engaged in the foundry business. His parents were of New England ancestry, natives of Vermont, who had emigrated to Lawrence County, New York, and, about 1836, had removed to Canada. When Mr. Frost was about a year old, his parents removed to Smith's Falls, Ont., where the boy obtained his early education at the village school. In 1854, he was sent to an academy in Glover, Vt., and on his return to Smith's Falls, he taught school while preparing for college. He was graduated from McGill University in 1860 with the degree of Civil Engineer, and, at the time of his death, was its oldest living alumnus.

After his graduation from McGill, Mr. Frost, as was the custom in those days, served his apprenticeship to a licensed land surveyor, until 1863, when he received his diploma as a Provincial Land Surveyor. His ambitions were not satisfied, however, by the opportunities offered him at his home in Smith's Falls, and, in August, 1863, he went to Chicago, Ill., where he secured employment as Rodman on a railroad survey then being made by the Chicago and Northwestern Railroad in Wisconsin. He remained with that Company, first in the office of the Chief Engineer and afterward in the Land Commissioner's office, until 1868, except for the summer of 1864 when he was employed in an architect's office in St. Louis, Mo.

After leaving the employ of the Chicago and Northwestern Railroad Company, in 1868, Mr. Frost engaged in the private practice of engineering with an office in Chicago. After the great fire in October, 1871, he did a great deal of engineering work in connection with the rebuilding of the city and in the establishment of property lines within the burned district. Many of the suburban sections which have been incorporated as part of the City of Chicago were laid out by Mr. Frost at that time. His work included also the survey of the tracts which now comprise the United States Reservation at Fort Sheridan and the Town of Glencoe.

In April, 1874, Mr. Frost established and issued the first number of the *Engineer and Surveyor,* a monthly publication devoted to the interests of the civil engineer, and the first periodical of its kind in America. In 1876, the publication was changed to a weekly and re-named the *Engineering News.* In December, 1878, Mr. Frost moved his business to New York City and opened an office in the Tribune Building. Later, he transferred his office to the St. Paul Building. The *Engineering News* steadily prospered under his management

* Memoir compiled by the Secretary, from information on file at the Society Headquarters.

until it became known as one of the leading authorities on civil engineering and allied subjects. In 1911, Mr. Frost sold his establishment to the Hill Publishing Company, of New York City, which recently was merged with the McGraw Publishing Company.

Mr. Frost always took great pride in the fact that he had carried the entire first few editions of his paper to the post-office himself, and that, during his 37 years of ownership, the paper had never missed an issue and was always out on time.

In 1886, Mr. Frost became a resident of Plainfield, N. J., and from the first took great interest in the local affairs of that place. When the City authorities were considering a sewerage system for the town, he took an active part in the work, serving, as a member of the Common Council, on the Committee on Streets and Sewers for five years. As Chairman of that Committee in 1893, he drafted the design for the sewerage system and devoted much thought, time, and labor to the project until its completion, in 1896.

After severing his connection with *Engineering News* in 1911, Mr. Frost retired from active business and devoted his time to study and travel. He was deeply interested in the study of genealogy and in geography and history, and he supplemented his reading by travel in many parts of the world. He had visited most of the countries of Europe and had made extensive trips to Egypt, South America, and in the United States. In 1914, he was planning a trip around the world, but gave it up on the outbreak of the European War.

Up to December 23d, 1916, Mr. Frost had enjoyed unusually good health. On that date, he suffered a stroke of paralysis and, although his condition improved for a time, he grew worse again, his death occurring at his home in Plainfield on March 15th, 1917.

On December 3d, 1868, he was married to Miss Louisa Hunt, a daughter of the late Edwin Hunt, of Chicago, Ill., and she, with four sons, survives him.

Mr. Frost was a thorough business man, with a capacity for the smallest as well as the largest detail. He was an optimist by nature, always looking on the bright side of things, his enthusiasm never lagging until his object was accomplished. Being deeply religious, he always took a keen interest in local church and charitable affairs, and was an active member of the Crescent Avenue Presbyterian Church of Plainfield.

Mr. Frost became a citizen of the United States in 1863, and was always afterward associated with the Republican Party. He was a member of the Canadian Society of Civil Engineers, and was also connected with a number of other scientific and technical organizations. At the time of his death, he was President of the Courier-News Publishing Company, of Plainfield, N. J., having purchased that paper in June, 1904.

Mr. Frost was elected an Associate of the American Society of Civil Engineers on January 4th, 1882.

JAMES JEROME HILL, F. Am. Soc. C. E.*

DIED MAY 29TH, 1916.

The death of James Jerome Hill, at St. Paul, Minn., on May 29th, 1916, removed from the railway, banking, and industrial life of the United States one of the most forceful and beneficent figures of his generation. Through the reflex action of his marvelous mind, as demonstrated in the many activities directly affecting the progress and welfare of the world, he became an international character, and will live in the future, in the influence and example of the sound policies and great works that his genius created during the busy years of his life.

In the vast circle of his friends, and among his business associates who knew him as he was, the demonstration of grief at his death was sincere and heartfelt, and the knowledge that such a personal and public loss was inevitable, in the course of Nature, made the sorrow no less poignant.

To condense even the main features of his history and life work within the limits of this memoir would be an impossibility, but, as a Fellow of the American Society of Civil Engineers, it is fitting that some record of his career and activities, and of the encouragement and co-ordination that he gave to the civil engineers who carried out his great works, should be made in the annals of the Society.

Mr. Hill was born near Guelph, Ontario, in the Dominion of Canada, on September 16th, 1838. His parents, James Hill and Anne (Dunbar) Hill, were of Scotch and Irish extraction, his father following the usual occupations of a farmer in a new country. His school education was obtained at Rockwood Academy, an institution of his home neighborhood, conducted by a Quaker schoolmaster, who exercised a strong influence on him and directed his reading and studies. His father died when he was fifteen years old, and he then took his share in the maintenance of the family by working in a country store near-by. His ambition, however, could not long be satisfied with such a narrow field, and three years later, in the summer of 1856, after visiting several of the larger cities of the United States, he settled in St. Paul, Minn.

As a young man without friends or resources, he found employment as a clerk on the river front, then the center of the business life of the little frontier settlement. He was first with the Dubuque and St. Paul Packet Company, then with the Davidson line of steamboats, remaining with that concern until he was appointed Agent of the

* Memoir prepared by Ralph Budd, W. L. Darling, and John F. Stevens, Members, Am. Soc. C. E.

Northwestern Packet Company in 1865. In 1867 he established a general fuel and transportation business on his own account, and in 1869 he became the head of the firm of Hill, Griggs and Company, engaged in the same enterprises. During all these years, by constant study of historical facts, human nature, and conditions, he laid the foundation of a business experience which came to include a knowledge of all the essential facts of the trade of the city and the country back of it, and began to estimate properly the wonderful resources of the latter, the possibilities of which had not even dawned upon our most far-sighted men.

The fuel business, even at that time, was important, and was closely allied with the restricted transportation facilities of the Northwest; but, during this period, Mr. Hill had become interested in the trade between the United States and Fort Garry, as Winnipeg was then called, and which was carried by way of St. Paul. To handle this business, he established, in 1870, the Red River Transportation Company, and in 1872 he started the first regular line of service between St. Paul and Fort Garry. The knowledge he thus gained of the vast latent resources of that country crystalized into one of the dominant ideas of his existence, and one which, extended and amplified beyond his wildest dreams, became an accomplished fact long before the close of his life. So that, as the result of his prophetic vision and tireless energy, an immense section of our country, which had been considered—if at all—of little value, has become a very potential factor in the production of the wealth of the world, and the homes of thousands of the best citizens of which the United States can justly boast. It was on one of his many trips, made in connection with this transportation business, that he met Donald A. Smith, afterward Lord Strathcona, with whom he was to be so closely and successfully associated in his first railroad enterprise.

There came to a troubled life in the Fifties, under the stimulus of legislation in Minnesota, a railway company called the St. Paul and Pacific, which name alone signified ambition, without knowledge or resources. It controlled valuable terminal lands in St. Paul, and had partly built a line, ultimately intended to reach the Canadian boundary at St. Vincent; but the road had failed, and had gone into the hands of a Receiver in 1873. Its bonds were held in Holland, by parties who were not willing to abandon their investment, nor had the foresight or nerve to invest more to salvage the wreck. Mr. Hill alone saw the possibilities of the road, and what a factor it could be made toward the development of the country, an empire then unsettled and overlooked. It was a ready forged but crude instrument, which, to be adequate for his purpose, must be controlled, properly financed, and made effective, all of which appeared to be a hopelessly visionary project, excepting to his master mind.

Its total debt was nearly $33 000 000, an enormous sum for those times, and the very size of this debt would have staggered a less optimistic and forceful character. He believed that he could secure, and bent all his energies toward securing, control of the property, and his faith was justified. The co-operation of Donald A. Smith, and of George Stephen (afterward Lord Mountstephen) was gained, and, together with Norman W. Kittson, who had had experience of the country in acting for the Hudson Bay Company, they formed a syndicate to buy the defaulted bonds of the companies covered by the St. Paul and Pacific Railroad Company.

The detailed story of the next few years reads more like a fairy tale than a relation of the sober facts of history. These four men pledged every thing they had in the world to obtain even money enough to secure an option on the bonds; for men of affairs in New York and London regarded the whole proposition as chimerical. Besides the securing of an option on the bonds, money had to be provided to finish uncompleted lines, within the time limit set by the Legislature, in order to avoid forfeiture; and all of this labor fell upon Mr. Hill, as did also the financing necessary to make good the agreements with the bondholders, until such time as the railroad could become a producer of net revenue, and it was not until 1878 that it could be truthfully said that matters really began to take proper shape.

In 1879 the properties were reorganized as the St. Paul, Minneapolis and Manitoba Railway Company, with necessary lines in operation, an organization created, the proper spirit infused into the enterprise, and a matured plan for the future mapped out. The system then comprised 555 miles of constructed road, and 102 miles under construction. From this time began its steady, never-ceasing expansion, covering the Northwestern country with new lines and branches, to be followed closely by immigration and settlement, and making an equally steady progress towards Montana, and the ultimate goal of the Pacific Ocean, which object had been in Mr. Hill's mind from an early day.

In 1888 the road reached Butte, and in 1893, the Pacific Extension touched the shores of Puget Sound at Seattle, Everett, and Bellingham, realizing the transcontinental idea. A line had long before been completed to the Great Lakes, and a steamship company established to give an eastern outlet.

Mr. Hill was for two years during this period interested in the planning and construction of the main line of the Canadian Pacific Railway, from the plains of Manitoba west to the Pacific Ocean, being Managing Director of that company; but in 1883 he gave up direct connection with that enterprise to devote his entire time to the welfare of his own marvelously developing properties.

He opened trade with the Orient, establishing a line of transpacific ships to develop and handle this new traffic. Locally, wherever his

lines touched, new industries sprang up; immigration and settlement, fostered by him as a personal matter, followed the rails with a rapidity hitherto unprecedented in the history of the United States.

In 1890 the various railroad enterprises which were offshoots of the original plan were all compacted into one company, the present Great Northern System. The great machine was pushed to completion, and grew bigger and better from year to year, until it stands to-day, in character and plan, as a marvel of transportation efficiency.

Mr. Hill was General Manager of the company from its beginning, became Vice-President in 1881, and President in 1882, which office he held and administered until he resigned it to one of his sons, just 25 years later. During all these years, as well as to the end of his life, his was the guiding hand, and his the inspiring vision and controlling mind that have made the Great Northern Railway not only the most notable long-distance transportation machine in the world, but also the greatest single factor in the material development of an empire of incalculable wealth of resources. This system, as the embodiment of the plans, ideas, and acts of one man, alone constitutes a fitting monument to the clear vision and untiring genius of James J. Hill.

As a student of railway economics, Mr. Hill had early known that the measure of success for a road was to be found in its ability to move traffic at the lowest practicable cost. In freight movement, this meant light gradients, with heavy power and large-capacity cars, with consequent maximum train loading. In this field he was preeminently the pioneer in American railroad practice. In the planning and construction of his lines, the cardinal principle, which he insisted must be followed, was that a dollar saved in construction is saved but once, but that a dollar saved in operation, is saved every time the wheels go over the track, and he saw to it that his engineers carried this principle out in every particular. Either by personal inspection or from reliable reports, he knew practically the resources and conditions of every mile of territory into which he projected his lines. He was naturally a great engineer, and always appreciated and gave credit where credit was due, to the different engineers whom he trusted to plan and execute the details of his projects. He quickly recognized merit and ability in the personnel of his engineering staff, and neither seniority nor rank influenced him in the least in preferment or promotions. He was equally quick to criticize and condemn, whenever his usually unerring judgment so dictated, and his attention to details together with his dynamic force of character kept every one of his staff up to the highest point of individual efficiency. But every engineer, however humble his rank, knew that, like Napoleon's soldiers, he carried a baton in his baggage, and the success and preferment of many engineers, not only of the Great Northern, but of other systems, were due, not only to Mr. Hill's forceful example, but also to his direct personal interest in their

welfare. Whenever he gave his confidence and support, he gave freely and fully. Results were what he demanded, and nothing pleased him more than to have an engineer leave the beaten track and find a new and better way to reach such results, and in this way individual initiative was encouraged. The Engineering Profession owes a large debt of gratitude to Mr. Hill, as some who were intimately associated with him for many years know full well, and appreciate.

As evidence of the great respect he had for the Profession, let his own words speak for themselves. In an address delivered before a gathering of Civil Engineers, at Minneapolis, Minn., on January 10th, 1908, he said:

"Yours is a great profession, following as it does the oldest of all the arts. When the pyramids were built there were engineers who mastered problems calling for great engineering skill. Relics of still older civilizations appear from time to time on the sites of cities whose names forgotten tell that the engineer was there also; and that he was, then as now, a leader of thought in matters of great practical service to the common good. The name of your occupation is significant, related as it is to the word 'genius' and implying the union of human ability with that diviner spark which kindles knowledge to a brighter flame.

"The engineer is, indeed, in no mean sense a creator. Not only does he provide for the needs of mankind those practical utilities on which civilization is based, but he brings into being structures, combinations, possibilities of dead matter and natural energy in relations new to men. He is inventor as well as artisan. Many of the modern wonders of the world are the thoughts of his brain as well as the works of his hands. The subjection of Niagara to our daily wants, the redemption of an ancient country by the damming of the Nile, great bridges such as span the Firth of Forth and our own East River at New York, tunnels that carry traffic under the estuary of the Severn and the peaks of the Simplon pass, tubes that enable railways to avoid waters too wide to be bridged safely, canals that cut away land barriers, the modern steamship that has cheapened and quickened ocean transit, the factories that have revolutionized industry and the marvelous plants which are now using the waterfalls of Norway to enrich the earth by the manufacture of fertilizers drawn from the nitrogen of the air—all these are conquests of the engineer's ability and bear tribute to him as the author of heretofore unthought of things. Across every chapter of the story of human development is written the symbol of some great engineer's creative mind."

The financial record of Mr. Hill's labors sufficiently attests to the sagacity of his ideas and their practical execution. While to superficial comment, it may be said that his individual pecuniary reward was great, the fact remains that he was never a money seeker for personal benefit. His whole ambition was to be a faithful custodian of the interests committed to his care, and no charge that he ever failed to make his work good can justly be made against him. His ability to

interest capital—domestic and foreign—in any project, or in line with any plan he set forth, was well known. He proved his faith and words by his acts, and such a course made him almost unique in the history of railroad finance in the United States.

As a characteristic act, his handling of the Great Northern iron ore interests may be mentioned. In 1899 he purchased an insignificant logging road, in northern Minnesota, which owned some lands. These lands contained iron ore, but in unknown quantities. At that time the vast deposits of such ore in that section had not been demonstrated, and his purchase involved a business risk in which he did not care to engage his stockholders. So the property was acquired as a personal investment. A company was formed to hold and develop the lands, which was done in the succeeding years, and when their immense value had been fully proven, they were all turned over and distributed to the holders of Great Northern stock at cost. What the present and potential value of the lands is no one knows, but it is a safe judgment that they alone are worth more than the personal estate which Mr. Hill acquired during his lifetime.

During all the busy years he gave to the creation and building up of the great system of railway, not a cent of salary was ever drawn by him from it, he preferring to take his compensation in common with the stockholders, from the increased value of the properties. Unlike other transcontinental lines, the Great Northern received no land grant, or other aid from the Federal Government, excepting the grant originally made to the St. Paul and Pacific Company in Minnesota. Its securities were marketable because they had value behind them, and from the further fact that the interests of the road were in the hands of a man who always had made, and was believed to be always able to make, his word good.

In 1901, the Great Northern and the Northern Pacific Railways jointly purchased the stock of the Chicago, Burlington and Quincy Railroad Company, thus giving to his other lines access to the enormous producing and consuming territory of the upper Mississippi Valley, a territory which for extent and variety of wealth-producing area probably cannot be duplicated in the world.

Mr. Hill had planned to place all the properties, in which his stockholders and himself were interested, in a position to be secure from future speculative attacks, and to ensure the harmony which efficiency demands, by uniting them in the Northern Securities Company. That company was dissolved by the decision of a divided Court, but the management of the separate companies, so far, has remained successful and harmonious.

Although the chief activities of Mr. Hill's life were expended in the development of railways, he did work enough along other lines to have more than sufficed to keep an ordinary man fully occupied. The

keynote of such work was his desire to raise the average level of prosperity, of intelligence, and of public spirit. Undoubtedly, the greatest pleasure he took in his work was in the efforts he made and the success he achieved in improving agriculture and in increasing its profits. He led the farmer away from the one-crop idea, and taught him to diversify his industry by stock raising. He imported, at large personal expense, stock of the most profitable strains, and distributed it, after practical demonstration on his own large experimental farm, among the most intelligent farmers of the Northwest, most of whom he knew personally. This practice he continued through thirty years, and the splendid condition of the live stock interest of that section to-day is largely the result of his labors.

He taught soil restoration and rotation of crops; set up a laboratory for soil analysis, and established demonstration plots, on farms along the line of the railway, to demonstrate to the farmers, by object lessons, what startling results could be obtained by scientific fertilization and cultivation. He was always ready to talk to a group of farmers, and gave no small part of his life's forces, through his later years, to the effort to persuade them to better methods.

He was the first to advocate the conservation of natural resources on a large scale. The memorable address which he delivered in 1906 created a profound impression, and ran through many editions and languages. He made heavy demands on his business time and physical resources to comply with requests to address bodies of representative men assembled to discuss economic and scientific problems, but he regarded this as a form of public service which it was his duty to perform.

Irrigation, waterways, all forms of internal improvement, banking and other forms of finance, the theory of credit, and the virtue of thrift, every aspect of railroad operation, regulation, and improvement, almost every practical human activity received his study and profited by the conclusion of his wonderfully analytic and almost unerring mind. Some of the more important of these were grouped in his volume of economical and sociological discussions, entitled "Highways of Progress", published in 1910. It is amazing to observe, in that collection, the range of his thought, the wealth of his information, and the validity of his conclusions.

Among the larger interests of his later years was the acquisition of the two banks in St. Paul, and their consolidation under the name of the First National, to which was joined the Northwestern Trust Company, and all placed under the immediate control of one of his sons. This great financial institution is intended to be, not merely a clearing house for the enormous financial interests of the properties in which he was concerned, but an agency for liberating the Northwest from outside control, and for increasing and stabilizing its prosperity.

It has been and is a power in advancing the interests of the farmers and helping them to surer and larger returns. Until a few days before his death, he was as busy as he had been for years before, with plans for the upbuilding of the country and the improvement of the common lot.

He was a patron and lover of the arts. His collection of pictures is known at home and abroad by its excellence, and is the result of his judgment in selection. He was a connoisseur of precious stones, and his investments in these were marked by his usual good business judgment. He was an omnivorous reader, and, being possessed of a remarkable memory, had accumulated a wealth of detailed information upon a wide range of subjects, such as few men of his age, or any age or generation, ever possessed. His philanthropy did not stop with words, for he was extremely generous, in a practical, material way. He gave impartially to many educational institutions, his only condition being that the schools that received his aid should be conducted under the auspices of some religious denomination.

His private benefactions were constant and great; they were made with care and discrimination, and he avoided publicity in them to the last degree.

For many years, during the earlier part of his career, he cannot be said to have attracted, excepting among his own group of business associates and capitalists, the more than passing notice of the then leaders of thought and finance. He never attempted to attract such notice by spectacular staging of any of his enterprises. His sayings and doings were rather looked upon by most eastern men with a mild commiseration; but, as his system of railways developed, and the results of his so-called theories began to prove their value, and to be seen in many respects to be the very basic foundation of the prosperity of the railways of the country, they were copied by other systems, and, from train haul to accounting, the Great Northern was for years a school at which railway men were glad to learn; and many graduates from that school, trained by practical experience in its methods, can be found holding high positions in railway service, in all parts of the United States. During the last decade of his life, there was no one whose voice, in general economics and financial matters, was listened to with more respect than that of Mr. Hill. The judgment of no one man in this country, perhaps, counted for so much in determining American financiers to take up the first loan asked by the Allies. He thought its acceptance essential to our own business life and prosperity, and that his reasons were sound and his conclusions well taken, is amply proven by subsequent developments.

Though his plans were well wrought out, and the results he achieved came from thorough preparation, he could strike with lightning quickness and terrific force, whenever occasion demanded. He

was plain of speech, simple and sincere of thought, but a giant of power when roused to effort by obstacles that had to be overcome, and he was—first, last, and all the time—an American. The men on his railroads recognized his character and ability, and through all classes and ranks there ran a spirit of cheerful loyalty that could only have been evoked by a man of his wonderful personality. Among the many who were privileged to know him intimately, particularly the old-timers, who had been with him in the long-gone-by days of his early struggles, the feeling was one of deep and sincere affection, and everywhere, over the whole nation and in other countries, the general sentiment of respect and sorrow at his death, as a personal and public loss, has been deep and wide.

It is a safe prediction that the great record of what he achieved will insure increased recognition as time goes on. Yale University conferred upon him the degree of LL.D. in 1910. The Hill Professorship of Transportation was established at Harvard not long ago, with an endowment contributed by seventy-four friends who were delighted to do him this honor.

He was a man of quiet domestic tastes, and his serenest hours were those spent in his beautiful home at St. Paul, where his friends were always welcome, and from which they always came away feeling that they had been privileged to share in the happiness of an ideal American fireside. Throughout all his years, he demonstrated in his own life that a strict adherence to moral laws was a true attribute of greatness.

Much of the planning of his construction work in all lines was done in the seclusion of his home, to which he habitually drew men of all classes for conference and instructions, and his library there was the workshop in which was produced many of his masterpieces.

On August 19th, 1867, he was married to Miss Mary Theresa Mehegan, of St. Paul, and his widow, three sons, and six daughters survive him.

Volumes of anecdotes could be written to illustrate his manner of dealing with various situations. Through them all runs the strong vein of humor which was so marked a feature of his character. Above all, he asked for independence in judgment by his subordinates, and no surer path to his disfavor could be found than to endeavor to forestall his opinion, and recommend accordingly. Though he was a man of strong convictions, even in matters of minute detail, he was always open to argument, and ready to receive and weigh suggestions, and to give them consideration. Once a decision was made, however, he expected speed and economy in the prosecution of the project in hand, and was content with nothing less than the best results. Take him all in all, to one who understood in a measure the salient features of his individuality, and realized the broad view-point from which he regarded

every subject, he was an eminently satisfactory man to be associated with, and such association was a liberal education.

After Mr. Hill's death, hundreds of messages of sympathy poured in upon the family from all parts of the country, from lands across the sea, from people of the greatest diversity of condition. It was a spontaneous tribute which showed in what esteem he was held by high and low alike. Corporations, associations, municipal bodies, cities, and States, united to regret him and to do honor to his memory. It is impossible to reproduce here any considerable portion of these tributes; but an extract, as of date June 3d, 1916, from the minutes of the Board of Directors of the Great Northern Railway Company, the creation of which was his greatest life work, shows to what degree he was appreciated by his close associates:

"*Resolved*, That in the death of James J. Hill, the Great Northern Railway Company laments the loss of the man who brought it into being, who made it an instrument for the development of this country, who led it with unerring wisdom, who foresaw and safeguarded its future, and to whom it owes all that it has been and all it hopes to be."

James Jerome Hill was elected a Fellow of the American Society of Civil Engineers on January 10th, 1889.

TRANSACTIONS

OF THE

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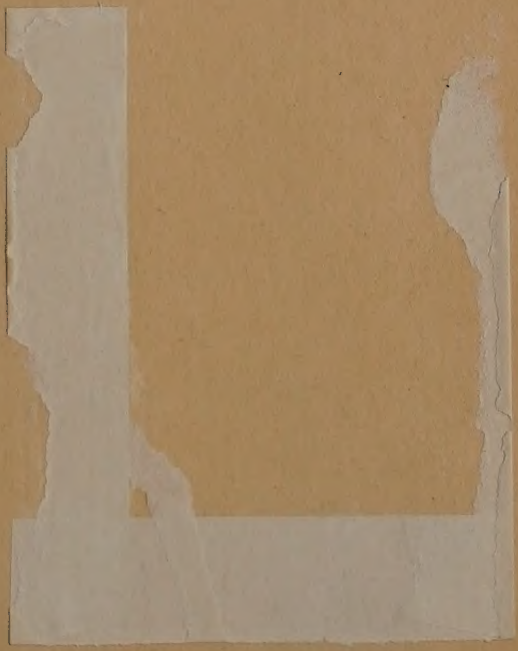
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